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CHAPTER I.

Classification of Roads.

Classification of Roads—Road Standards.

CLASSIFICATION OF ROADS.

1. Military roads in India are classified under the following general heads :— General Heads.

Roads inside cantonments,
Roads outside cantonments,

and are designated Imperial Military Communications.

2. Military roads inside cantonments comprise roads inside barrack, etc., areas, and approach roads from main roads or railway stations primarily or exclusively serving such areas. They also include portions of Military Trunk Roads within cantonment limits (see below). Military Roads inside cantonments.

3. Roads generally, except within or approaching Military areas, are Civil Communications, constructed and maintained from P. W. D. or other civil funds. Civil Roads.

Subsidiary roads in cantonment areas not classified as Military are, as a rule, constructed and maintained from cantonment funds. Cantonment Fund Roads.

4. Military roads outside cantonments are roads classified as such for strategic or special administrative reasons. They comprise principally Military Frontier roads and roads leading to exclusively or primarily military stations or military defences, arsenals and factories, etc. Military roads outside cantonments.

5. For military and administrative reasons, selected Imperial Civil communications may be classified by the Government of India as roads of Military Importance. Such roads will be maintained by the Military Engineer Services (or when so classified at the time of construction, will be constructed by the Military Engineer Services). The expenditure necessary on their upkeep or improvement (or construction) will be met from Civil funds transferred to the Military Engineer Budget for the purpose, to the extent required for their maintenance or improvement (or construction) to the standard required for Civil purposes. In the event of standard required for Military purposes being higher than that necessary for Civil purposes, all expenditure over and Roads of Military Importance.

above that necessary for the Civil standard will be financed from Military funds.

Roads of Military Importance comprise principally certain Civil Frontier roads.

ROAD STANDARDS.

Road Standards.

6. Roads are divided into two main classes :—

Metalled Roads,

Unmetalled Roads,

Military roads in India (subject to variations inside cantonments) are normally classified :—

Metalled or Unmetalled.	Class.	Standand for traffic.	Short description.
M	I	Suitable for "continuous" M. T. traffic.	24′ roadway, metalled 16′ wide, fully bridged (*i.e.*, bridged except where traffic would not be impeded by the use of causeways).
M	II	Suitable for "intermittent" M. T. traffic.	20′ roadway, metalled 12′ wide, partially bridged (*i.e.*, bridged across perennial streams, and where traffic would be impeded across causeways for more than 24 hours; causeways otherwise provided in lieu of bridges wherever practicable and cheaper).
M	III	Suitable for "occasional" M. T. traffic.	18′ roadway, metalled 9′ wide, unbridged except across perennial streams (causeways provided elsewhere in lieu of bridges, wherever practicable and cheaper).
U	IV	Suitable for carts, and in certain cases for "occasional" light M. T. traffic, principally in dry weather only.	12′ roadway, unmetalled, and generally unbridged.
U	V	Suitable for Camel traffic.	10′ roadway, unmetalled, and unbridged.
U	VI	Suitable for pack mule traffic.	8′ roadway, unmetalled, and unbridged.

On account of the development of mechanical transport, Class IV roads will now seldom be constructed for military purposes; lowest standard of permanent road constructed for wheeled transport will usually be Class III.

Classes V and VI roads are practically exclusively hill roads.

Temporary Roads.

7. In the above remarks permanent roads are referred to. Temporary roads constructed by troops on active service are not specifically dealt with therein; the standard of such roads will vary according to local conditions and requirements, but the principles of alignment, gradients, and drainage should be the same.

CHAPTER II.

General Specifications for metalled roads.

Introductory—Specifications—Special Methods of Road Surfacing—Road Survey.

INTRODUCTORY.

1. To specifications and dimensions, etc., here given are of primary application to military lines of communication or roads outside cantonments. They are, however, of equal application to roads inside cantonments, subject to local differences such as increases or variations in roadway width, or in metalling width, provision of foot paths and kerbs, complete bridging or provision of culverts in all classes, etc. Application of specifications.

2. A summary of normal leading dimensions, etc., is given in Appendix I. Leading dimensions.

3. When constructing a new road, the land width, alignment, gradients, etc., will be such that the road can be converted from the lowest into any of the higher classes specified, if required. Convertibility.

4. In constructing a road, the essential items to permit of traffic getting through as early as possible should usually be carried out in the first instance. These include adequate drainage. Metalling should be done when the roadway and drainage have been completed in individual miles or sections. Order of work in construction.

SPECIFICATIONS.

5. On irrigated or cultivated land, or flat plains :— Land width.

Normal—60′.

On barren hill sections.—The acquisition of a definite land width is generally impracticable or unnecessary. Sufficient ground should be taken up to allow for catchwater drains where necessary, and for stacking metal off the roadway.

Through villages or cemeteries, where the full width is impracticable, the maximum practicable width should be taken up.

6. The inner edges of borrow pits should be not less than 10′ from the foot of the slope of embankments. Along very high embankments the land width may, therefore, have to be increased. Borrow pits should not be continuous except where used as roadside drains. Borrow pits are strictly forbidden in cantonments; any earth required must be brought from outside. Borrow pits.

Demarcation of road boundaries.

7. By furlong stones on both sides.

Between boundary stones by the edges of borrow pits or drainage ditches.

Gradients.

8. Ruling.—1 in 20.

In long stretches of ruling gradients, flats 100′ to 200′ long should be given every half mile approximately.

Maximum.—1 in 13, over distances not exceeding 300′. On curves of 100′ radius or less the gradient should not exceed 1 in 20.

The gradient of the centre line throughout a sharp curve should be nil or very slight.

The rise per mile should not exceed 240′.

Radius of curves.

9. Centre line of roadway.

Normal minimum.—60′.

Minimum at zig zags in hill portions.—35′.

For methods of setting out curves, see Plate II.

Blind corners and zig zags.

10. As far as possible, blind corners must be mitigated by cutting back the inner side to 4′ above road level, to secure visibility.

They should also be indicated by the standard road sign, placed on either side.

A low circular dry stone wall should be provided within the inner drain, to keep vehicles from short circuiting the curve.

(See also Gradients, Super-elevation, Road Signs).

Super-elevation on curves.

11. All except very slight curves should be suitably cross-banked. The pitch of the banking should vary inversely with the radius of the curve, to a maximum of 1 in 7 for a curve of 35′ radius.

The slope must be continuous across the full formation width and the cross-banking must be carried across culverts and small bridges located on sharp curves, allowance being made for this in designing and building parapets and wheel guards in such cases.

In construction, particularly where the road is to be used for traffic before the formation has settled, the super-elevation should be exaggerated.

Width of roadway.

12. Clear width at formation level, excluding side drains, and parapet walls :—

	Class I.	Class II.	Class III.
Normal	24′	20′	18′
In straight through or side cuttings in hard rock	20′	18′	16′
On full embankments exceeding 2′ in height, and in sharply curved and blind through or side cuttings, or sharply curved approaches to bridges and causeways . . .	27′	24′	22′
In tunnels	18′	18′	12′

Long deep through cuttings or tunnels are to be avoided particularly in shale or alluvial soil.

13. Up to 2′ high.—1 in 4, to permit of motors getting off if necessary. **Outer slopes of embankments.**

Over 2′ high.—Natural slope of soil.

14. Subgrade (*i.e.*, the surface on which the soling is laid). **Subgrade.**

Height in hill sections.—Nil.

Height in plains sections.—Normal minimum 6″.

It is essential to keep the subgrade dry, to prevent settlement. In flats where the side drains are apt to be full of water for long periods, and on bad ground generally including saturated or black cotton soil the subgrade should be raised to 18″ or more above ground level.

On bad boggy ground, or deep black cotton soil which cannot be removed, where an alignment over such ground is unavoidable, special foundation measures are necessary and in such cases standard works on the subject should be consulted.

In long very gentle slopes with good and frequent outfalls for the side drainage, the subgrade may remain at ground level.

Whether the subgrade is raised or not, it should always be dressed, depressions filled up, and rolled to the formation camber or cross-slope, before soling is laid; embankments must be thoroughly consolidated.

15. Side drains should be provided on both sides of the roadway except in side cuttings in hill sections, where they are to be given on the inner side. **Roadside drains.**

The minimum width of side drains is 2′, minimum depth 15″.

In hill sections the side drain may be triangular in section, the bottom sloping off in continuation of the berm, at a slope of 1 in 3, so that vehicles may not fall into the drain.

In soft formations in hill sections, where the drains are necessarily small, the side drains should be lined with dry stone where practicable.

16. In hill sections catchwater drains should be provided, except on loose soil or shaly hill sides. They should be constructed alongside the roadway, at a distance above the roadway varying from 10′ on steep slopes to 40′ on gentle slopes, and should be connected at intervals to the cross drainage. **Catchwater drains.**

Catchwater drains, except in very hard formations, should be lined with large stones or brushwood to prevent erosion.

17. Cross drainage must be adequately provided for. **Cross drainage.**

In hill sections scuppers (small paved crossings) may be used instead of culverts, where practicable, for minor cross drainage.

An average of 8 culverts or scuppers per mile will usually suffice for minor cross drainage in hill sections, but the requirements must be fully catered for in each case.

Rain water must be given a clear flow both above and below all cross drainage, in the direction in which it is desired to make it go.

Diverging boulder bunds are often useful, above cross drains in hill sections, to direct the flow.

Drainage on steep hill roads with zigzag turns in particular requires special study and attention, to ensure the drainage of the upper portions being directed to the cross drains, below.

Drainage and flood protection for the roadway must be constructed concurrently with the road formation.

(See also Culverts, Scuppers, and Metalled Dips.)

Soling. 18. Soling must always be provided, except where unnecessary upon rock or hard conglomerate.

The function of soling is to spread concentrated loads over the subgrade.

Width of soling :—

Normal :--1 foot more than the metalling.

On high sandy banks or close to high bridge abutments, and in metalled dips. - Full width of clear roadway.

On culverts and bridges metalled throughout, where soling is required. Width of metalling.

Thickness of soling. Before consolidation.

Normal : –Using quarried hard rock or large boulders.—6".
Using small boulders or softish material.--9".

On embankments and other soft subgrades, particularly where the subgrade cannot always be kept dry, and in metalled dips. 9" to 12".

Soling should consist of the hardest suitable material procurable locally. Large stones, kankar, or overburnt brick bats are suitable. If round boulder stones are used they should not be less than 5" in any dimension.

Soling must be hand-packed, and interstices filled with chips as the stones are laid. When quarried hard rock or large boulders are used, a layer not exceeding 3" of small shale, local stones, shingle, or gravel, should be laid on the top.

The soling (including the top layer in the case of hard rock or large boulders) should be rolled with a steam roller, depressions being filled up as they form with shingle or gravel, sufficiently only to give a hard true surface, barrelled or sloped to the correct camber or pitch.

In cases where earth only is available locally for the surfacing, it may have to be used, but earth should not be used unless unavoidable. In any case, depressions must be filled up with broken metal or gravel, etc., never with earth.

A road may be opened to light traffic, including light M. T. as a temporary measure, pending metalling, subject to the sub-

grade where soling is not required having been finished off as specified, and to the soling being covered with a 2″ layer of gravel and re-rolled, before traffic passes. The metalling should be done as soon as possible.

Where the soling is extended outside the metalling on high banks, etc., it should be similarly gravelled and rolled; the finished surface in such cases must, of course, conform to the slope of the metalled surface and the unsoled portion of the berms.

19. Width of metalling :— Metalling.

	Class I.	Class II.	Class III.
Normal	16′	12′	9′
On curves of 100′ radius or less having a subtended angle of 45° or more . . .	20′	16′	12′
On bridges and culverts	Full width.		

Wherever practicable, a 16′ width of metalling should be given through villages, in all classes.

Thickness of metalling :—

	Class I.	Class II.	Class III.
Layers before consolidation.	2 × 4½″	1 × 6″	1 × 4½″

The thickness of metal, after consolidation, should be 6″ 4″ and 3″ in Class I, II and III roads, respectively.

In cases where earth has unavoidably to be used for surfacing soling, the thickness of the metal must be increased, thus :—

	Class I.	Class II.	Class III.
Layers before consolidation	2½ × 4½″	½ × 4½″ 1 × 6″	1½ × 4½″

Metalling must not be commenced until the soling (or dressed subgrade where soling is not given) has been properly finished and passed by competent authority.

Road metal should be comprised of hard, tough, and durable broken stone. Soft stone, such as sand stone, unless thoroughly indurated by the action of heat, must not be used for metalling, and flint should not be used as it is difficult to consolidate. The best all round stone procurable for this purpose is usually quarried limestone. Where boulder stone has to be used, the boulders must not be less than 5 inches in diameter, and must be well broken to eliminate round surfaces as much as possible.

The gauge of the metal ballast should normally be from 1½" to 2", according to the hardness of the stone, and the weight of the steam rollers available, the smaller gauge being adopted for the hardest stone, and particularly for boulder metal.

All layers of metal should be laid and consolidated separately.

Consolidation comprises :—

(i) Spreading and rolling dry.
(ii) Watering and rolling wet.
(iii) Surfacing and rolling wet.

Where two coats are to be given, the third process should be omitted in the case of the first coat (unless the road has to be opened for traffic before the second coat is provided).

The metal should first be dry rolled until set. The amount of dry rolling necessarily varies with the quality and thickness of the metal, but as a general rule two thorough rollings with a 12-ton steam roller will suffice. It should then be flooded, and thoroughly rolled until hard and compact so that a light cart makes no impression. The metal must be freely flooded during consolidation. The screenings from the metal, supplemented as necessary by gravel, fine shale, or sand, should then be spread on the surface, watered, and thoroughly rolled in. The surfacing should not exceed ½" in thickness before rolling.

Finally a top dressing ½" thick of river sand or fine gravel should be spread on the surface, and the road should be opened to traffic at once if possible; it should be kept watered under traffic for about a fortnight, and the surface should be specially inspected daily and breaks repaired with water, bajri, and rammers during this period.

Sometimes the only hard stone available at reasonable cost (*e.g.*, basalt or flinty boulder metal) cannot be consolidated without the use of a binder of the best small material locally available. Such cases require special study and orders, as the resulting roadway is not true macadam, but a kind of gravel or mud concrete. Apart from such special cases no binding material whatever may be used during the first two processes of consolidation.

As a temporary measure, where water is procurable with difficulty, and it is desired to open the road to traffic emergently, dry consolidation may be adopted; in this case the surface, after consolidation, must be covered with 2 inches of gravel, bajri, or fine shale, rolled in. In such cases the metalling must be reconsolidated with ample water as soon as possible.

For further details regarding metalling, see Chapter X.

Collection of soling and metalling and metal stacking places.

20. Soling stones and metal ballast should not be collected a long time before they are required, as they become dispersed and mixed with dust and earth.

Except where absolutely impracticable, they should be stacked off the roadway.

Soling stones must not be placed on the roadway until the subgrade has been properly excavated or filled, and graded.

Particularly in hill sections, metalling must not be placed on the roadway until the soling has been laid.

Metal stacking places for repair metal must be provided in hill sections, where level ground alongside the road is not available. These may be at intervals of 2 furlongs, as practicable.

21. (Crown slopes). See Plate II. Camber of road surface.

Normal, *i.e.*, in straights:—1 in 40.

On curves:—No camber, super-elevation cross slope to be given.

In construction, particularly where the road is to be used for traffic before the formation has settled, the camber should be exaggerated.

22. Width of berms:—Clear width of roadway on either side outside metalling, excluding space taken up by road parapets and side drains:— Berms.

	Class I.	Class II.	Class III.
Normal	4′	4′	4½′
Range, according to widths of metalling and of clear roadway.	2′—5½′	2′—6′	3½′—6½′

The overall widths of berms, including the space occupied by parapet walling on embankments, and by the roadside drains in hill sections, will be wider than the above figures by 2′ to 3′.

Slope of berms:—

Normal, in straights:—1 in 40.

On curves:—No camber, super-elevation cross slope to be given.

Berms should be graded off in continuation of the camber of the metalling (on banked curves to the cross slope). (See Plate II).

In soft soil the berms should be consolidated and dressed to correct slopes before soling is laid, as otherwise they act as sponges and prevent the subgrade from drying.

When made up after metalling has been completed, they should be rolled to correct slopes, in continuation of the metalled surface. A layer of gravel (shale, shingle, or rough stones), should be rolled in on the clear roadway portion of earthen berms. This is particularly necessary on slopes, in through cuttings, and on banked curves, and in other cases should not be omitted unless surfacing material is not procurable at site.

For further details see Chapter X.

Culverts.

23. Culverts (total length between abutments not exceeding 12′).

Width:—The clear width between wheel guards should equal the clear width of the roadway, *i.e.* :—

	Class I.	Class II.	Class III.
Normal	24′	20′	18′
Minimum	20′	18′	16′
Maximum	27′	24′	22′

Culverts in hill sections up to 4 feet span can frequently be economically and efficiently built of dry stone. The cover or decking is made of stone slabs, or reinforced concrete, or the sides are corbelled out and slabbed over. At least 12″ of filling (earth, soling, and metalling, see also Chapter VI) should be placed over the slabs.

Tube culverts provide an economical and quickly constructed form of small culvert, where such is suitable. The tubes may be made of ordinary corrugated iron, not thinner than 18 S.W.G., and of 2′ or 2½′ diameter, surrounded by 1 to 2 feet of good lime concrete, lime concrete filling being provided between tubes in series, or they may be made of reinforced concrete, or of specially manufactured rust resisting corrugated iron tubes (such as " Armco "), which do not require a lime concrete surround.

Tube culverts should extend across the full formation width. Each end must as a rule be completely protected by masonry so that the water does not form a false channel outside the tube.

Culverts must always be constructed in one span, except in special cases where the use of a series of tubes is suitable.

The minimum clear span for a culvert is 2′, and this is only permissible when the drainage is off rock and free from heavy silt or large boulders. Culverts should be too large rather than too small.

Where the outfall is not on to rock, special precautions must be taken to prevent erosion of the foundations of the drop or retaining wall.

The bottoms of culverts in hill sections must be given the steepest possible slope, not less than 1 in 12, and may have to be floored to prevent erosion or percolation.

Culverts in hill sections must be well sunk, to avoid humps in the road, and must be provided with adequate catchpits, the bottoms of which should be a foot or more below the floors or sills of the culverts.

All culverts should be soled (where soling is necessary), and metalled, full width between wheel guards.

On flat ground, the slopes of approaches to culverts should not exceed 1 in 25, and a length of 20′ over each culvert should be flat, the tops and bottoms of slopes being rounded off, to avoid humps in the road.

For details of culverts, see Chapters V, VI and VII.

24. Scuppers. (Paved dips not exceeding 20′ in span.) Scuppers

Width :—Formation width.

Scuppers should be freely used, where practicable, in class III roads. They should not be used as a rule in class I roads.

Scuppers should have a normal cross slope of 1 in 12 in hill sections to 1 in 30 on the flat and should be located on re entrant curves in hill sections, never on salient curves.

They must cross the road at right angles and must be carefully graded in cross section to avoid sharp dips in the road, their edges being rounded off.

They are often preferable to culverts in hill sections, in soft shaly formations, and also where large boulders are likely to come down, and small culverts would readily become blocked.

They are unsuitable for use on steep gradients.

Where the outfall is not on to rock, special precautions must be taken to prevent erosion of the foundations of the drop or retaining wall.

For details of Scuppers, see Chapter VIII.

25. Width :— Metalled Dips.

	Class I.	Class II.	Class III.
Clear width of roadway	..	20′	18′

Supplementary cross drainage in very flat country is often provided for satisfactorily in second and third class roads by dropping the formation down to ground level over wide gaps, the width being made sufficient to prevent appreciable afflux occurring, with consequent dangerous velocity. In such cases the full width of formation must be soled 9″ to 12″ thick, and occasionally it may be necessary to build a continuous drop wall at the downstream side of the soling. The approach slopes must not exceed 1 in 14, and their top and bottom ends must be rounded off. The admixture of a little cement with the metalling gives a finished road surface proof against minor erosion.

26. (Minor bridges:—Total length between abutments not exceeding 100′. Bridges.

Major bridges :—Total length between abutments exceeding 100′.)

Width :—

	Class I.	Class II.	Class III.
Clear width between wheel guards . .	18′	18′	10′

Special orders regarding bridging are issued in the case of each road.

As a general rule, bridges must always be provided across perennial streams, except in minor instances.

In class I roads causeways may be substituted for bridges where traffic across them would not be liable to interruption.

In class II and III roads bridges may be freely replaced by causeways, under the limitations laid down for causeways.

On class III roads the piers and abutments of bridges should be built as for 18′ bridges, permitting of widening. Arched bridges should usually be built to 18′ width, in all classes.

In the case of small bridges located on sharp curves the clear roadway width should be widened 4′.

All bridges should be soled (where soling is necessary), and metalled, full width between wheel guards.

Approaches to bridge on the flat should not be steeper than 1 in 25.

Bridges should be designed to span streams with a minimum of restriction.

Sharply turned approaches to bridges should be widened and cross banked, in accordance with the rules for sharp curves.

For details of bridges, see Chapters V, VI and VII.

Strength of bridges and culverts.

27. All bridges and culverts should be designed to carry a 12-ton steam roller, with 25 per cent. impact.

Bridges so designed will safely take any train of loads in which no axle load exceeds 8 tons and no distributed or caterpillar axle load exceeds 12 tons.

They will also take any train of loads in which no axle load exceeds 12 tons and no distributed or caterpillar load exceeds 18 tons, with precautions (minimum intervals, speed not to exceed 4 miles per hour, load on centre of road, and no other load alongside).

Thus they will safely take a double line of 3-ton loaded lorries, 15-ton caterpillar tractors at not less than 30′ intervals, all marching formations of cavalry, infantry, field artillery, and animal transport, and crowds of pedestrians and animals.

They will also take heavier vehicles with precautions, including 16-ton wheel tractors and 18-ton whippet tanks at 25′ intervals.

For further details and calculations, see Chapter V.

Causeways.

28. Width :—

	Class I.	Class II.	Class III.
Overall	18′	18′	12′

Causeways should be used instead of bridges in class II and III roads where practicable and cheaper than bridges, and where traffic across them would not be impeded for more than 24 hours and at infrequent intervals. They should only be given in place of bridges in class I roads where traffic across them would not be liable to interruption by floods. Causeways are unsuitable where the maximum depth of scour exceeds 6 feet, and in any case should not be provided if their cost with approaches is more than $\frac{2}{3}$rds of the cost of a bridge with approaches.

The end slopes should be normally 1 in 14, to a height of 1½′ above high flood level. The top and bottom of the slopes must be rounded off.

The causeway surface should slope down-stream at the same slope as the nala bed, subject to a maximum of 1 in 30. The length of a causeway should be such as to span the width of the stream at high flood level.

In the case of wide nalas, in which two or more streams flow, these should, where practicable, be trained through one causeway of length equal to the aggregate width of all the stresses at high flood level. Causeways should be at right angles to the current, and should not restrict the stream.

Sharply turned approaches to causeways should be avoided; where unavoidable they must be widened and cross-banked in accordance with the rules for sharp-curves.

For details of causeways, see Chapter VIII.

Overflow Bridges.

29. Width :—

	Class I	Class II.	Class III.
Overall	18′	18′	12′

Overflow bridges may be used, if cheaper than ordinary bridges, instead of, or in conjunction with, causeways, subject to the limitations laid down for the use of causeways. They are unsuitable for torrential nala crossings in the hills, and where heavy silting occurs.

For details of overflow bridges, see Chapter VI.

Parapets and Hand Rails.

30. Height. Above road surface, normal :—

In roadside parapet walling	2′
On culverts	1½′
On bridges	2¼′

Parapet walls should be provided on both edges of all banks over 10′ high.

They are usually made 2′ thick, of rough dry stone, in lengths of 6′-20′, with 1′-2′ gaps, or large rocks may be used.

The height of parapets or hand rails of long high bridges may be increased to 3′.

The parapets or hand rails of bridges and culverts must be protected by wheel guards or kerbs, and fenders.

Wheel guards should be 6″ wide, by 9″ high above the road surface.

Fenders, consisting of stout posts or rails well embedded in the ground, or large rocks, should be provided at both ends of parapets or hand rails, to a height of 2′ above the road surface.

In trans-frontier districts masonry or reinforced concrete parapet walls or hand rails should be given on bridges and culverts, in preference to iron hand rails, which are liable to destruction and removal by tribesmen.

For details of bridge and culvert hand rails and parapet walls, see Chapter VI.

Head room in Tunnels and Over-bridges.

31. Over centre 8′ of roadway :—13′.
Over sides of roadway : 8′ (minimum).

Retaining and Breast Walls.

32. Retaining walls, except where impinged upon by floods and generally where a high degree of strength is necessary, will usually be constructed of dry stone masonry.

In high dry stone walls a band of stone in lime should be given at intervals varying with the quality of the stone. A normal method is to give a 12″ band, every 6′, in walls over 10′ high.

The top thickness is usually 2′, the front batter 4 over 1, and the back face should be vertical. In breast walls the front slope should not exceed 2 over 1. All courses should be normal to the front batter, as also the foundations, which should be laid in lime concrete where the soil is unfavourable.

It is generally advisable to bed each course in very fine shale or earth, to spread the load and increase the frictional resistance between courses, particularly where shale slabs are used.

Pucca retaining walls will of course be given concrete foundations.

Incorrectly designed and constructed retaining walls are a great source of weakness in a hill road, and special care must be taken that specifications and designs for them are efficient and complete.

Details of retaining walls are given in Chapter IV.

Training Works.

33. The main principle in regard to training works is that they should lead or guide the water, in contradistinction to forcing it.

Projecting spurs are as a rule unsuitable for torrential hill

streams, except as guide bunds. Guide bunds designed to protect crossings (bridges or causeways) should be sited well upstream of the crossings.

Where restriction is unavoidable at a crossing, or where a causeway or bridge spans a portion of a nala between high banks at a site where it would be manifestly extravagant to span the entire width between high banks, any necessary protection should be designed with the specific object of guiding the water straight through the crossing. Particularly in the case of major bridges, this protection is best given by providing a bund at either or both abutments. running upstream of the site, and at right angles to the bridge, to a distance upstream equal to the length of the bridge.

Except in the case of a silting river, it is frequently advisable, particularly in dealing with torrential hill streams, gradually to curve off the bund above this point, at an angle of 120° to 140°, to the pucca high bank, into which it should be firmly built.

Where necessary, the abutment guide bunds described above should be carried a short distance down-stream of the abutments, and curved off at the end.

All projecting bunds must be firmly built into the pucca bank at the point where they take off from it. Exposed terminal ends of all bunds should be curved round, and their foundations should be particularly well protected against scour.

For further details regarding training works, see Chapter VIII.

34. Care must be taken to construct foundations of bridges, causeways, and culverts of adequate depth to avoid destruction by scour. **Scour and depth of foundations.**

A useful rough rule, applicable to torrential nala beds of boulders and shingle, is that the depth of scour in straight runs, where no restrictions to waterway are caused by the structure, is equal to half the maximum depth of water at high flood level, and on curves it is equal to the maximum high flood depth. Foundations should be not less than twice the depth of scour in the case of culverts and causeways, increased to three times in the case of bridges. Each case must of course be considered on its merits, with due regard to the formation of the nala bed as well as other considerations of catchment area, slope, velocity, etc.

For details, see Chapter V.

35. Sidings (halting or passing places) will ordinarily be provided at watering places, at the top of all long inclines, and at each end of bridges which are not wide enough for double wheeled traffic. They should be capable of taking three lorries, and should be located, if possible, alongside level stretches of road. **Sidings and Parking Places.**

Normal dimensions :—66′ in length by 16′ maximum width, the outside edge away from the road to be on the arc of a circle of 39′ radius.

Sidings should be soled and metalled throughout, in continuation of the roadway metalling.

They should be provided with an outer perimeter drain connected to the roadside drain, except where unnecessary in the case of sidings sited on the outer sides of half cuttings, or of embankments.

Metalled parking places of suitable sizes should be provided if specially ordered, at termini and stages.

Levelling and gauges. 36. Boning rods should be used in preparing the subgrade to get even slopes between points where the gradient changes.

In hilly country the centre line should be pegged out by an experienced officer and all considerable curves should be accurately aligned with the aid of a theodolite, when they cannot be struck from a centre. Subsequently the edges of the soling and metalling must be demarcated with strings from the centre line.

Accurately constructed wooden gauges with plummets must be used to get the proper slopes for camber, slopes for berms, cross banking, batter of walls, etc.

Mile and Furlong Stones. 37 The standard type of milestone is triangular in section with a sloped triangular top. The distances to important places in either direction are given on the two corresponding main faces, together with the months in which the adjacent miles were last remetalled. On the top face are recorded the record mileage (*i.e.*, measured from the start of the road), and the altitude.

In trans-frontier districts, to avoid temptation to destruction of regular milestones by tribesmen, the mileage and furlongage is frequently painted on large rocks.

Furlong stones should be provided on both sides of the road to mark the road boundaries on cultivatable land.

For design of mile and furlong stones, see Chapter IX.

Direction and Warning Signs. 38. Direction and warning road signs are grouped under the following heads :--

Road Direction Posts,
Warning Signs and Notices,
Village and place name signs,
Road name signs.

Signs, of the standard patterns, should be provided, as necessary.

In the case of roads outside cantonments in trans-frontier districts regular sign posts may be liable to destruction by tribesmen. In such cases road signs should be provided to the extent practicable ; thus warning signs, conforming to the standard devices and colours, may be painted on large rocks.

For designs of road signs, and standard symbols, see Chapter IX.

Roadside trees. 39. Roadside trees will be provided, if specially ordered, in plains sections.* They should be planted outside the road formation, but inside drainage ditches and borrow pits, except in the case

of high embankments, where they may have to be outside them, if provided. The normal minimum distance of the lines of trees from the céntre of the roadway is 20.'

SPECIAL METHODS OF ROAD SURFACING.

40. The use of oil and tar spraying, tar macadam, of cement concrete, for road surfacing, is not here specified, as owing to climatic conditions, the difficulty of obtaining materials of adequate or suitable quality, and expense, these methods are not at present practicable on Military roads in India, as a general measure. Tar macadam or spraying, and oiling, are however desirable in cantonments and in M T yards, to improve the wearing capacity and to minimize dust, and the use of these methods should be developed as they are perfected by experience for Indian climatic conditions. A reference to these methods is made in Chapter X. Special Methods of Road Surfacing.

No measure of special surfacing will, of course, entirely remove the dust or mud nuisance on roads on which the provision of full width metalling or other surfacing cannot be afforded.

ROAD SURVEY.

41. Practical notes on traversing and levelling are given in Volume I, Chapter X. Road Survey.

Particulars of the maps, road plans, and cross sections required, are given in Chapter XI of this Volume (Estimating).

Methods of setting out curves are given on Plate II.

Instructions regarding levelling and gauges are given in para. 36 of this Chapter.

CHAPTER III.

General specifications for unmetalled Roads.

Introductory—Unmetalled Cart Roads—Pack Transport Roads.

INTRODUCTORY.

Application of specifications.

1. The specifications and dimensions, etc., here given are of primary application to military roads outside cantonments.

They are, however, of equal application in principle to roads inside cantonments, in cases in which unmetalled roads are required.

Leading dimensions.

2. A summary of normal leading dimensions, etc., is given in Appendix III.

UNMETALLED CART ROADS.

Unmetalled Cart Roads.

3. The specification for an unmetalled cart road (Class IV). should follow that for a metalled road, subject to the omission of soling and metalling, and other variations indicated. The alignment should usually be such that the road can be improved to a metalled standard if required.

Super-elevation.

4. In unmetalled cart roads (Class IV), super-elevation is not necessary on curves.

Road Surface.

5. In localities where shale or good gravel is found the surface should be dressed and rammed or lightly rolled to the required formation, and the traffic will work in the material and produce a hard surface.

In softer ground a 2″ layer, 9′ to 18′ wide of broken stone or brick ballast, covered with 1″ of clay, and lightly rolled in, over 6″ to 12″ of loose stones, may be given where practicable and desirable.

On hard rocky ground, the surface should be dressed to camber, etc., depressions being filled in and rammed.

Scuppers and Causeways.

6. Scuppers and causeways will as a rule be freely used, where practicable, instead of culverts and bridges, in Class IV roads.

Sidings.

7. These are not provided on Class IV roads, except where specially required in hill sections.

Road Signs.

8. Road signs, except mile and furlong stones, direction posts, and village signs, are not usually provided in Class IV roads except at very dangerous places.

PACK TRANSPORT ROADS.

9. Camel and mule roads (Classes V and VI) belong to a different category, and are not aligned so that they can be subsequently converted into a wheeled traffic road, unless specially ordered. They are primarily hill roads. **Pack Transport Roads.**

10. The ruling dimensions of these roads are given in the summary of leading dimensions for unmetalled roads (Plate II). **Ruling dimensions.**

The general instructions in the specifications for metalled roads are of applieation to Class IV and V roads in the following cases :—

Land width, drainage, retaining and breast walls, training works, levelling and gauges, and bridges and culverts as applicable.

11. The road surface must be dressed to camber or cross slope and rammed if necessary. **Road Surface.**

12. The maximum gradients of 1 in 8 and 1 in 6 for Class V and VI roads are permissible in lengths not exceeding 400 and 300 feet, respectively, and the rise per mile must not exceed 500 and 750 feet, respectively. In Class V roads, on curves of radius less than 20 feet, and in Class VI roads, on curves of radius less than 10 feet, [illegible]ent should be nil or very slight. **Gradients.**

13. The surface drainage on gradients should be intercepted by shallow rough dry stone cross drains. These cross drains should be provided at intervals of 50′ to 150′ according to the steepness of the gradient: the steeper the gradient the smaller being the intervals between the drains. **Drainage.**

On zig-zags particular care must be taken to build the cross drains on the lower stretches of the road to correspond with those higher up, and every opportunity should be taken to lead the drainage right away from the road at all bends.

14. Culverts are not usually provided in Class V and VI roads, except where essential across irrigation channels. **Culverts.**

Where they are necessary dry stone culverts or tubes are generally suitable.

15. Bridges and causeways are not provided in Class IV and V roads, except where bridges are specially necessary, as across a gorge to avoid a devious alignment, or across a deep perennial stream. **Bridges and Causeways.**

16. Sidings and super-elevation are not provided in Class V and VI roads. In hill sections these roads should ordinarily be given a cross slope of 1 in 20-30 to the inner drain, or to the outside edge on re-entrant curves. **Sidings and Super-elevation.**

17. Mile and furlong stones should only be provided in Class V and VI roads where specially ordered. **Mile and Furlong Stones.**

18. Direction posts and village signs may be given in Class V and VI roads, if specially ordered. **Direction and Warning Signs.**

19. Roadside trees are not provided in Class V and VI roads. **Roadside trees.**

CHAPTER IV.

Retaining walls.

Introductory—Principles of Design—Graphical Method of Design—Analytical Method of Design—Breast Walls—Economic Sections for Retaining Walls—Methods of Failure and General Remarks—Specifications for Retaining Walls.

INTRODUCTORY.

Introductory.

1. General particulars regarding the design and construction of retaining walls are given in the General Specifications (Chapter II).

In this Chapter detailed methods of design are described. These principles are of equal application to abutments, which are essentially retaining walls.

PRINCIPLES OF DESIGN.

Natural Data.

2. *Weights.*

Rammed earth	100	lbs. per c. ft.
Sand, dry	100	,, ,, ,,
Sand, moist	100	,, ,, ,,
Sand, wet	110–125	,, ,, ,,
Clay	110–130	,, ,, ,,
Gravel and Sand	120–130	,, ,, ,,
Water	62½	,, ,, ,,
Lime Concrete	115	,, ,, ,,
Brickwork	120	,, ,, ,,
Dry stone masonry	130	,, ,, ,,
Cement Concrete	130–150	,, ,, ,,
Stone Masonry	160	,, ,, ,,

Angles of repose.

Earth, loose	30°–45°
Earth, rammed	60°–70°
Sand, dry	25°–35°
Sand, moist	30°–45°
Sand, wet	15°–30°
Clay, dry	25°–30°
Clay, damp or well drained	30°–45°
Clay, wet	15°–20°
Gravel and Sand	25°–40°
Water	0°

Co-efficients of friction.

Masonry upon moist clay	·33
„ sand	·40
„ dry clay	·50
„ gravel	·60
„ masonry	·70

The angle of internal friction is about 20 per cent. to 10 per cent. greater than the angle of repose of the loose material; 25 per cent. may be used generally.

3. Units are degrees, feet, and pounds. Notation.

θ=angle of internal friction of the backfill material.

Σ=angle of surcharge of the backfill.

μ=angle of resultant force on the wall to the vertical.

h=vertical height from base to top of filling on a vertical line through the heel.

H=height of wall.

B=width of base without toe projection.

T=toe projection beyond face of wall.

C=ratio of horizontal to vertical pressure.

P=total horizontal pressure on wall.

F=total of vertical forces.

R=resultant of P and F.

V=total vertical load per foot length of wall from the vertical component of the backfill pressure (P tan Σ) and any superimposed load (*e.g.*, bridge span).

w=weight of backfill material in lbs./c. ft.

m=weight of wall material in lbs./c. ft.

$$k=\frac{m}{w}$$

p=toe pressure on base B in lbs./sq. ft.

f=toe pressure on (B+T) in lbs.

n=face batter (*e.g.*, for a batter of 4 over 1, n=4).

$\frac{1}{q}$=foundation slope (*e.g.*, for a slope of 1 in 8, q=8).

e=excentricity of the resultant.

=the distance from the centre of the base at which the resultant cuts the base.

N.B.—For convenience, the forces on one foot length of wall are considered throughout.

4. See Plate V.

The direction of the resultant pressure of the backfill material on the wall is always parallel to its surface. The horizontal and vertical components of the resultant pressure are P (see para. 3) and P tan Σ, where Σ= the angle of surcharge of the backfill material. Backfill pressure.

The intensity of the horizontal pressure on a retaining wall depends on the unit weight w of the filling, and the ratio c of the

horizontal to the vertical pressure. c depends upon the values of the angle of internal friction θ and the angle of surcharge Σ.

1 f c=·25, and w=100.

The horizontal pressure=c w =25 and the vertical pressure=w =100,

i.e., for vertical pressures the material may be considered as a fluid weighing 100 lbs./c. ft., and for horizontal pressures as a fluid weighing 25 lbs./c. ft. (For water c=1).

The horizontal pressure per sq. ft. at any depth H=c w H . (1)
and the total pressure for any depth H, or against a wall of height H,

$$P=\frac{c\,w\,H^2}{2}\ \text{(Rankine)} \quad . \quad . \quad . \quad . \quad (2)$$

The location of P is through the centre of gravity, *i.e.*, $\frac{H}{3}$ above the base.

(i) With a level backfill, when $\Sigma=O$,

$$c=\frac{1-\sin\theta}{1+\sin\theta} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3a)$$

(ii) With an angle of surcharge Σ between O and θ,

$$c=\cos^3\Sigma\,\frac{\cos\Sigma-\sqrt{\cos^2\Sigma-\cos^2\theta}}{\cos\Sigma+\sqrt{\cos^2\Sigma-\cos^2\theta}} \quad . \quad . \quad . \quad (3b)$$

(iii) With an angle of surcharge $\Sigma=\theta$

$$c=\cos^2\theta \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3c)$$

A graphical method of finding c is also given on Plate V.

For an increase in Σ from O to 4/5 θ, c increases by about 1/6, but as Σ increases from 4/5 θ to θ, c rapidly increases, until when $\Sigma=\theta$. c is 2 or 3 times its value corresponding to $\Sigma=\theta$.

From this it can be deduced that wing walls at 45° to the direction of the slope of a surcharge fill can be designed as for a level fill, without appreciable error.

Effect of a super imposed load.

5. If a load W, such as a vehicle or a building, is placed on the fill within a distance H of the top of the retaining wall, the wall must be designed for an increased height

$$H^1=H+\frac{W}{H\,D\,w} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

where D=the length parallel to the wall on which the superimposed load W is distributed.

Thus, if W=a 12 ton road roller

H=14 ft.

D=15 ft.

w=100 lbs./c. ft.

$$H=14+\frac{12\times2240}{14\times15\times100}=15{\cdot}28 \text{ ft.},$$

and this value must be used for H.

6. The forces acting on a retaining wall, which are to be considered, are — Forces on a retaining wall.

(i) Horizontally,

P = the total horizontal pressure of the backfill.

(ii) Vertically,

$$F = W_m + W_b + P \tan \Sigma + W_s \quad . \quad . \quad . \quad . \quad . \quad (5)$$

Where W_m = the weight of the masonry of the wall.

W_b = the weight of the backfill between the back of the wall and a vertical line through the heel.

$P \tan \Sigma$ = the vertical component of the backfill pressure. (= 0 in the case of water or a level backfill).

W_s = any superimposed load, such as the superstructure and live load on a bridge in the case of abutments.

The resultant of P and F is R, acting at an angle μ with the vertical and cutting the base at a distance e (excentricity) from its centre.

7. For safety against overturning, the resultant R of the horizontal and vertical forces must not pass outside the middle third of the base of the wall. When the resultant cuts the base at the minimum distance of ⅓ of the base from the toe, the pressure on the heel of the wall is zero and on the toe it is twice the average; the factor of safety against overturning is then between 2 and 3. — Overturning.

8. The pressure on the foundation material at the toe of the wall must not be greater than the safe working stress. By projecting the toe the centre of the base is brought nearer to the resultant, and the toe pressures are very much reduced (see Plate VI). Projections on the heel increase the maximum toe pressures, and should not be given. — Pressure on foundations.

The average pressure on the foundations

$$P_a = \frac{\text{total vertical force or load}}{\text{width of Base}}$$

$$= \frac{F}{B+T} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (6)$$

The difference in pressure at the toe and heel due to the excentricity e of the resultant force from the centre of the base

$$P_e = \frac{6 \times \text{vertical force} \times e}{\text{Base}^2}$$

(See Volume I, Chapter IV and Plate IV, Case A).

$$= \frac{6 \times F \times e}{(B+T)^2} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (7)$$

The maximum pressure at the toe,

$$P_t = P_a + P_e \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (8)$$

and at the heel, $P_h = P_a - P_e \quad . \quad . \quad . \quad . \quad . \quad . \quad (9)$

If P_e is greater than P_a, giving tension at the heel, which is impossible, the maximum toe pressure at the toe

$$P_t = \frac{2 \times \text{vertical force}}{3\,x\,b}$$

where $x = B+T \qquad (T+e)$

and b = length of wall under consideration $= 1'$.

(See Volume I, Chapter IV and Plate IV, Case B).

$$i.e.,\ P_t = \frac{2\,F}{3\,x} \qquad \ldots\ldots \qquad (10)$$

Sliding. 9. The wall must have a factor of safety of 2 against sliding.

To meet this condition, $\tan\mu$ must not exceed half the co-efficient of friction of the wall on the foundation material.

The value of $\tan\mu$ can be decreased, if this proves necessary, by sloping the foundation downwards towards the backfill (see Plate VI), and taking this foundation slope as 1 in q, $\tan\mu - \frac{1}{q}$ must be less than half the co-efficient of friction.

For the graphical method of design

$$\tan\mu = \frac{\text{total horizontal force}}{\text{total vertical force}} = \frac{P}{F} \qquad (11)$$

For the analytical method,

$$\tan\mu = \frac{c\,w\,H^2}{2\,V +} \qquad (12)$$

where V = vertical load per ft. length of wall, from the vertical component of the backfill pressure and any superimposed load (*e.g.*, a bridge).

B = width of base without toe projection.

p = toe pressure on base B in lbs./sq. ft.

Explanation of Diagrams. 10. Plate IV illustrates an abutment solved graphically, utilizing the graphical solution of c shown on Plate V.

The tables on Table VII give values for use in the analytical method of solution of retaining walls.

Plate VI gives standard proportions for retaining walls, to suit the average conditions stated thereon. These proportions can be applied direct in practice, subject to the requisite calculations being made in all important and exceptional cases. (See also para. 18.)

Graphical Method of Retaining Wall Design.

First Example (25' abutment). 11. See Plate IV, also Plates V and VI.

An abutment has been chosen, as it introduces a load from the bridge. The design as a retaining wall would be on the same principles, omitting the bridge load.

A masonry abutment 25′ high is to be designed to retain an earth backfill and to carry a 100 ft. bridge span. The maximum pressure on the foundations must not exceed 2 tons=4,480 lbs./sq. ft.

The proportions given for masonry abutments with a vertical face will be adopted as a first trial, and if not satisfactory will be modified.

See Chapter VI and Plate XVII, 3rd type.

Width of base without toe projection,

$$B = \cdot 40 \times \text{height } H = 10 \text{ ft.}$$

Width of toe projection,

$$T = \frac{H}{10} = \frac{B}{4} = 2\cdot 5 \text{ ft.} \quad \text{Make } T = 3 \text{ ft.}$$

The forces on one foot length of wall will be considered.

The weight of the filling

$$w = 100 \text{ lbs./c. ft.}$$

The weight of the wall material

$$m = 150 \text{ lbs./c. ft.}$$

$$K = \frac{m}{w} = 1\cdot 5$$

Angle of internal friction $\theta = 45°$

By the graphical method given on Plate V, [Case (1)], plotted on Plate IV, the ratio of horizontal to vertical pressure $c = \cdot 17$ or, by equation (3a), (para. 4),

$$c = \frac{1 - \sin\theta}{1 + \sin\theta} = \frac{1 - \sin 45°}{1 + \sin 45°} = \cdot 17$$

By equation (1), (para. 4), the horizontal pressure 25 ft. from the surface **Horizontal Force.**

$$= cwH = \cdot 17 \times 100 \times 25 = 425 \text{ lbs./sq. ft.}$$

The whole pressure may be represented graphically by the triangle abd (Plate IV).

$$P = \frac{ab \times bd}{2} = \frac{25 \times 425}{2} = 5{,}320 \text{ lbs.}$$

or, by equation (2),

$$P = \frac{cwH^2}{2} = \frac{17 \times 100 \times 25^2}{2} = 5{,}320 \text{ lbs.}$$

P is taken as acting through the centre of gravity of the triangle, *i.e.*, at a height $\frac{H}{3} = 8\cdot 33$ ft. above the base.

By equation (5), the vertical component of the resultant force on the wall, per ft. length of the wall, **Vertical Force**

$$F = W_m + W_b + P \tan \Sigma + W_s$$

$W_m = W_{m\,1} + W_{m\,2}$, the weights of the rectangle and triangle which may be taken as composing the abutment. (See Plate IV).

$$W_{m1} = (25 \times 3{\cdot}3 \times 150) = 12{,}400 \text{ lbs.}$$

$$W_{m2} = \frac{(25 \times 6{\cdot}7 \times 150)}{2} = 12{,}600 \text{ lbs.}$$

$$W_b = \frac{(25 \times 6{\cdot}7 \times 100)}{2} = 8{,}400 \text{ lbs.}$$

$$P \tan \Sigma = O$$

For W_s, the weight of the 100 ft. span is about 3,000 lbs. per ft. run. The abutment will be 20 ft. long.

$$\therefore W_s = \frac{3{,}000 \times 100}{2} \times \frac{1}{20} = 7{,}500 \text{ lbs.}$$

$$\therefore F = 12{,}400 + 12{,}600 + 8{,}400 + 7{,}500 = 40{,}900 \text{ lbs.}$$

The horizontal distances from the vertical face of the wall, of the forces making up F, are :

$$W_{m1} \ldots\ldots\ldots \frac{3{\cdot}3}{2} = 1{\cdot}65 \text{ ft.}$$

$$W_{m2} \ldots (3{\cdot}3 + \frac{6{\cdot}7}{3}) = 5{\cdot}53 \text{ ft.}$$

$$W_b \ldots\ldots (3{\cdot}3 + \frac{2 \times 6{\cdot}7}{3}) = 7{\cdot}77 \text{ ft.}$$

$$W_s \ldots\ldots\ldots\ldots\ldots\ldots\ldots 1{\cdot}25 \text{ ft}$$

The moment of F about the face of the abutment = the sum of the moments of its components.

$\therefore$ the distance of F from the vertical face

$$= \frac{(12{,}400 \times 1{\cdot}65) + (12{.}600 \times 5{\cdot}53) + (8{.}400 \times 7{\cdot}77) + (7{.}500 \times 1{\cdot}25)}{40{,}900}$$

$= 4{\cdot}02$ ft.

Resultant. To find the resultant, plot to any suitable scale the triangle of forces mnq, in which

mn = F = 40,900 and nq = P = 5.320.

Then R = mq = 44.200 lbs.

Overturning. The resultant R intersects the base ·55 ft. beyond the centre, *i.e.*, within the middle third, which satisfies the condition of stability against overturning

Pressure on foundations. By equation (6), (para. 8), the average pressure on the foundation

$$P_a = \frac{F}{B+T} = \frac{40{,}900}{13} = 3{,}140 \text{ lbs./sq. ft.}$$

By equation (7), the difference in pressure at the toe and heel due to the excentricity e of the resultant R,

$$P_e = \frac{6 \times F \times e}{(B+T)^2} = \frac{6 \times 40{,}900 \times {\cdot}55}{13^2} = 800 \text{ lbs./sq. ft.}$$

i.e., 800 lbs. compression at the toe and 800 lbs. tension at the heel.

By equations (8) and (9)

Max. pressure at toe

$p_t = p_a + p_e = 3{,}940$ lbs./sq. ft.

Max. pressure at heel

$p_h = p_a - p_e = 2{,}340$ lbs./sq. ft.

The diagram of base pressures is drawn as shown on Plate IV. The rectangle 1, 2, 3, 4 represents the average pressure $= 3{,}140$ lbs./sq. ft.

The differences due to excentricity (800 lbs./sq. ft.) are plotted as 3, 5 and 4, 6.

Triangle 6, 7, 4 cancels out, leaving as the resultant diagram of base pressures 1, 5, 6, 2.

The angle of the resultant R with the vertical, $\mu = qmn = 7\frac{1}{2}°$ Sliding.

$\therefore \tan \mu = \tan 7\frac{1}{2}° = \cdot 13$.

For masonry on earth, the co-efficient of friction $= \cdot 40$

Tan μ is less than $\frac{1}{2}$ this.

$\therefore$ the abutment is safe against sliding.

As the resultant lies within the middle third of the base, and the maximum pressure on the foundation, and the resistance to sliding, are satisfactory, the design is suitable. Conclusion.

12. If the maximum foundation pressure were excessive, the toe would require more projection. If the resistance to sliding were too little, the base width of the wall would have to be increased, or the base would have to be sloped down towards the backfill at a slope $1/q$ such that $\frac{P}{F} - \frac{1}{q}$ is less than the safe value for tan μ ($\cdot 20$). Variations.

13. It should be noted that, if there were no toe projection, the maximum toe pressure would be as follows :- Necessity for toe projection.

$$e' = \left(\frac{B+T}{2} - \frac{B}{2} + e\right) = (1\cdot 5 + \cdot 55) = 2\cdot 05, \text{ say } 2 \text{ ft.}$$

Average base pressure

$$p_a' = \frac{F}{B} = \frac{40{,}900}{10} = 4{,}090 \text{ lbs./sq. ft.}$$

By equation (7), (para. 8), the difference in pressure due to excentricity

$$p_e' = \frac{6 \times F \times e'}{B^2} = 4{,}900 \text{ lbs./sq. ft.}$$

$\therefore$ max. pressure at toe

$= p_a' + p_e' = 8{,}990$ lbs./sq. ft.

and p_e' being greater than p_a' there would be a theoretical tension at the heel

$= p_a' - p_e' = -810$ lbs./sq. ft.

$\therefore$ by equation (10), in this case the maximum toe pressure would be

$$p_t' = \frac{2F}{3x}$$

where $x = \frac{B+}{} - (T+e) = 6{\cdot}5 - 3{\cdot}55 = 2{\cdot}95$ f

$$\therefore p_t' = \frac{2\times40{,}900}{3\times2{\cdot}95} = 9{,}250 \text{ lbs./sq. ft.}$$

This is double the permissible maximum load on the foundation, and shows the necessity for the toe projection in this regard.

ANALYTICAL METHOD OF RETAINING WALL DESIGN.

Analytical Method.

14. The graphical method of design is tedious, and may necessitate several trials before the right section of wall is determined. The graphical method need only be used for special cases not easily solved analytically.

The analytical method is much quicker, and gives directly the necessary proportion $\frac{B}{H}$ and the other requisite details.

In Table VII, equations for determining the value of $\frac{B}{H}$, together with tabulated results for $\frac{B}{H}$, maximum toe pressures p, and factors of safety against overturning, to meet various conditions, are given.

The equations for $\frac{B}{H}$ are of equal application to walls retaining water, where $e=1$.

Second Example (30′ retaining wall).

15. A retaining wall is to be designed 30′ high to retain a level earth backfill with an angle of internal friction = 37°. The maximum pressure on the foundations must not exceed $1\frac{1}{2}$ tons = 3,360 lbs./sq. ft.

A wall with a vertical back will be adopted. Make the top thickness of the wall 2′.

$H = 30$ ft.

$\Sigma = 0$

$\theta = 37°$

$K = \frac{m}{w} = \frac{150}{100} = 1{\cdot}5$

$e = \frac{1-\sin 37°}{1+\sin 37°} = {\cdot}25$

From equation (7) on Plate (T. 1)

$\frac{B}{H} = \sqrt{\frac{}{K}} + {\cdot}01 - {\cdot}05 = {\cdot}37$

$\therefore B = {\cdot}37 \times 30 = 11{\cdot}1$ ft. Say 11′ 1″.

From the tables on Table VII,

Max. toe pressure

$p = 190\ H = 5{,}700$ lbs./sq. ft.

$$\frac{\text{Max. allowable pressure}}{\text{Max. pressure on base B}} = \frac{3{,}360}{5{,}760} = \cdot 59$$

$\therefore$ a toe projection T is necessary, such that the toe pressure on the base (B+T), f, is not greater than ·59 p.

From the foundation pressure table on Plate VI, $T = \frac{B}{5}$ gives $f = 56$ p.

Make $T = \frac{B}{5} = 2\cdot 22$ ft. Say 2′ 3″.

From Table VII, the factor of safety against overturning, for base B, $= 2\cdot 06$; this is increased by the effect of $\frac{T}{2}$.

For sliding, by equation (7), (para. 4),

$$\tan \mu = \frac{cw\ H^2}{2V + pB}$$

$$= \frac{\cdot 25 \times 100 \times 30^2}{0 + (5{,}700 \times 11\cdot 1)} = \cdot 355$$

But $\tan \mu$ must not exceed

$$\frac{\text{co-efficient of friction}}{2} = \frac{\cdot 40}{2} = \cdot 20$$

$\therefore$ the base must be sloped at $\tan^{-1}(\cdot 355 - \cdot 20)$
$= \tan^{-1}\cdot 155$, or 1 in $6\frac{1}{2}$.

From the above results, and the proportions for retaining walls given on Plate VI, the design can be completed

16. The foundation pressure at the toe, in the case of the 25′ abutment worked out in para. 11 and solved graphically on Plate IV, can be approximately found analytically as follows :—

Application of analytical method to 1st Example.

The excentricity e_s of w_s

$$= \left(\frac{B+T}{2} - T - 1\cdot 25\right) = 6\cdot 5 - 3 - 1\cdot 25 = 2\cdot 25$$

By equations (6) and (7), the toe pressure due to

$$W_s = \frac{W_s}{(B+T)} + \frac{6 \times W_s \times e_s}{(B+T)^2} = \frac{7{,}500}{13} + \frac{6 \times 7{,}500 \times 2\cdot 25}{13^2} = 1{,}17?$$

lbs./sq. ft.

From the table on Table VII corresponding to $K = 1\cdot 5$ and equation (1) thereon for a vertical faced wall, the toe pressure due to the remaining vertical forces, (for base B, disregarding the toe projection),

$p = 260\ H$ approx.
$= 260 \times 25 = 6{,}500$ lbs./sq. ft.

The toe projection $T = 3' = \frac{3B}{10} = \frac{B}{3\cdot 3}$

Interpolating this in the foundation pressure table on Plate VI, the correct toe pressure f, on (B+T) $=\cdot 39$ p approx.

$=2{,}540$ lbs./sq. ft.

$\therefore$ Max. toe pressure $=1177+2540=3717$ lbs./sq. ft.

This may be compared with the value 3,940 lbs./sq. ft. derived by the lengthy graphical process

For sliding,

$$V = P \tan \Sigma + W_s = W_s$$

and by equation (6)

$$\tan \mu = \frac{cwH^2}{2V+pB}$$

$$= \frac{\cdot 17 \times 100 \times 25^2}{(2\times 7{,}500)+(6{,}500\times 10)} = \cdot 133$$

which practically agrees with the value for tan μ, $\cdot 13$, obtained by the graphical method

Breast Walls.

Breast Walls.

17. Proportions for breast walls are given on Plate VI, Types E and F. The equation for $\frac{\text{Base}}{\text{Height}}$ in this case is

$$\frac{B}{H} = \sqrt{\frac{c_1}{1\frac{1}{3}\,K.} + \left(\frac{1}{2n}\right)^2} - \frac{1}{2n}$$

where n = the batter (usually = 3/1 or 4/1) and

$c_1 = 2/3$ c for a batter of 3/1,

or $= 3/4$ c for a batter of 4/1.

Max. toe pressure $p = 125$ H.

For sliding, $\tan \mu = \frac{c_1 H}{2\,(V + B.\,H.\,K.)}$

Breast wall designs are solved similarly to those for ordinary retaining walls, subject to the above special equations applicable to them.

Economic Sections for Retaining Walls.

Economic Sections.

18. The relative quantities of material, and the toe pressures for walls of various sections, taking a rectangular cross section as 1·00, are :—

Face vertical.	12/1	8/1	6/1	4/1	Back vertical.	Rectangular wall.	Symmetrical wall.
Material . ·69	·62	·61	·60	·59	·57	1·00	·61
Toe pressure ·97	·81	·77	·74	·67	·63	1·00	·75

The rectangular cross section requires the most material, and also produces the largest toe pressures on the foundation.

For simplicity in construction and for economy in material and low base pressures, the sections with face vertical or a batter of 12 over 1, or with the back vertical, are the best.

METHODS OF FAILURE AND GENERAL REMARKS.

19. Retaining walls usually fail through errors in construction such as :— Methods of Failure.

(i) Settlement of the toe, due to toe shallow foundations or to the presence of water at the toe, or to no toe projection being provided to reduce the pressure on the foundation.
(ii) Lack of proper drainage, allowing the fill to become saturated and causing a large increase in the earth pressure.
(iii) Carelessly placed backfill not properly rammed.
(iv) Insufficient bonding in the masonry courses.
(v) Masonry courses not at right angles to the face of the wall.

An examination of the proportions for retaining walls according to Marryat's specifications, and Trautwine's, General Fanshawe's and Sir Benjamin Baker's practical rules, shows that they agree with the proportions derived by theory for an angle of internal friction between 45° and 53°.

The retaining walls for average conditions shown on Plate VI are based on $K=1{\cdot}25$ and $\theta=45°$.

If local experience from failures indicates that these walls are not strong enough, their proportions may be increased, provided that the failures are not due to lack of proper foundations or drainage, to sliding or erosion, or to excessive toe pressures on the foundations due to an inadequate toe projection on the base.

In all important and exceptional cases retaining walls must be specially designed by the methods herein described.

20. For a surcharged slope it is sufficient to increase the base dimensions by 20 per cent. In such cases the resultant earth pressure is parallel to the surcharged slope of the fill, and although it is greater than for a level fill it has a smaller leverage, and the overturning moment is not much greater, except when θ is less than about 30°, (a rare occurrence except with water and very wet sand, when special designing is always necessary). Surcharged Slopes.

SPECIFICATIONS FOR RETAINING WALLS.

21. According to the strength and service required, retaining walls may be constructed of :— Specifications.

(i) Rough dry stone, on lime concrete foundations if necessary.
(ii) Rough dry stone, with strengthening bands of stone in lime.

(iii) Masonry in lime or lime concrete, on lime or cement concrete foundations.

(iv) Masonry in cement or cement concrete, on cement concrete foundations.

(i) and (ii) apply to ordinary retaining and breast walls, not impinged upon by floods and not required to hold water.

(iii) and (iv) apply to abutments, and to important retaining walls or portions thereof. (See also Chapter II, para. 32.)

The specifications generally should comply with the masonry specifications in Chapter VI and Volume I, as applicable.

Foundations. 22. Foundations must be taken deep enough to reach solid material, safe from scour, frost, and surface water. Rock must be cut in level steps or to a downward slope towards the filling.

The foundations must be at least 1 foot, plus 1/10 of the height of the wall, below ground level

Masonry. 23. The base must be substantial and be capable of distributing the pressure over the foundation. The projection of any footing course should not exceed half the depth of the course.

Masonry courses must be normal to the face batter; the back of the wall can be left rough or stepped. Special attention must be given to through bonding, specially in dry stone walls.

Coping. 24. The top thickness of the wall should not be less than 1′ 6″ or 2′ 0″. The coping should consist of large stones, which must be laid in cement mortar where liable to damage from traffic. The top of the coping should be weather sloped.

Drainage. 25. Adequate provision must be made to prevent water accumulating behind the wall. Weep holes 2 to 3 inches square should be provided about every 6 feet vertically and horizontally, the lowest being about one foot above ground level.

The inlets of all weep holes should be surrounded by loose stones. In wet situations a continuous loose stone drain should connect the horizontal weep holes.

Backfill. 26. The layer of backfill immediately against the wall should consist of stone or of the most granular material available. The remainder of the backfill should be rammed in 6 inch layers, sloping away and downwards from the back of the wall.

CHAPTER V.

Bridges and culverts—Calculations for waterway, Depths of foundations, and loads.

General Principles of Design—Calculations for Waterway and Depths of Foundations—Calculations for Load on Bridges and Culverts on Mechanical Transport and Cart Roads—Calculations for Load on Bridges and Culverts on Pack Transport Roads.

GENERAL PRINCIPLES OF DESIGN.

1. As a general rule, it is not economical or advisable to restrict the waterway when constructing bridge or culvert, except in very exceptional cases. A restricted waterway increases the velocity and the scour, and the consequent safe depth of the foundations; it also necessitates training works which will often be of an elaborate and expensive nature, above and frequently also below the bridge. *Waterway.*

2. Foundations must always be taken down deep enough to be safe from scour. No reliance should be placed on training or protection works to reduce the depths of piers and abutments. *Foundations.*

3. The alignment of all training bunds, abutments, piers, banks, and retaining walls in the river bed in the vicinity of a bridge must conform to the current rather than the road. Thus a stream line alignment should be presented by piers, etc.; training works should be aligned to lead the water rather than force it; and generally speaking all preventable recesses or projecting points, which would cause eddies, cross currents, be likely to cause additional scour, must be avoided. *Alignment of bridge works.*

4. Culverts should always be designed in one span (except where the use of a group of tubes is suitable and economical). *Numbers and lengths of spans.*

As a general rule, spans of bridges should be as large and as few in number as possible.

For each type of bridge, there is a certain economic span length, which will give the minimum total cost, and which should be considered with due regard to practical considerations.

5. The span of minimum cost occurs when the cost of one span of the structure carrying the load direct to the piers is equal to the cost of one pier. *Rule for Economic Spans.*

Items which are common to a large or small span such as metalling, kerbs, parapets, handrails, and floor slabs, in beam bridges, and floor slab and beams in truss bridges, do not enter into the problem, as their cost per foot of span does not vary with the length of the span.

Take for example a series of R. S. Joist spans with a slab floor, total height of piers 12 ft. Then the economic span will be about $12 \times 1\frac{3}{4} = 21$ ft.

The comparative costs of the piers and R. S. Joists for 10, 20 and 40 ft. spans of height above bed level up to about 12 ft., are approximately as follows :—

NOTE.—The floor, etc., is common to all spans and does not enter into the comparison.

Clear span	10 ft.	20 ft.	40 ft.
Cost of 1 pier	7	8	10
Cost of 1 span of R. S. Joists .	2	8	32
TOTAL .	9	16	42
Cost per foot of span . . .	·9	·8	1·05

Also for the same clear waterway the smaller spans will require a greater total length of bridging and the floor system will be longer in the proportions of 115 per cent., 110 per cent. and 107 per cent., respectively.

There is therefore no economy in relatively short spans ; they require more piers, which multiply the difficulties met with in the foundations and they cause greater scour and obstruction to the stream.

General Proportions for Economic Spans.

6. Approximately the economic span is as follows :—

Masonry arches	L=2 H or more.
Reinforced concrete slab on masonry piers . .	$L = 1\frac{1}{2}$ H.
Reinforced concrete slab on pile bents . .	$L = \frac{3}{4}$ H to H.
Steel troughing, R. S. Joists and reinforced concrete beam spans on masonry piers . .	$L = 1\frac{3}{4}$ H.
Steel Truss Spans on masonry piers . . .	$L = \frac{3}{2}$ H.

Where L = span length in feet centre to centre of piers for minimum cost, and
H = total height of one pier or pile bent, from the underside of the foundation to the top of the pier or pile bent, and for arches to the intrados of the keystone.

The cost will not vary appreciably for $\frac{2}{3}$ to $1\frac{1}{2}$ times the above values of L, and for twice these values the cost will increase by about 25 per cent.

Where the foundations must be very deep and expensive, it will probably be cheaper in the end to use spans of twice the economic span, and halve the number of piers required.

Special rules for Masonry Arches.

7. For economy in material in the arch ring and abutments, the rise of the arch should be between $\frac{1}{2}$ and $\frac{1}{4}$ of the span, and the span should be not less than the height in the case of culverts, or not less than $1\frac{1}{2}$ times to twice the height in the case of bridges. The height is measured from the underside of the foundations to the underside of the keystone.

Spans in which the rise is less than ¼ of the span, or in which the height is greater than the span, require more material in abutments, piers, and wing walls.

For the same area of opening it is more economical to increase the span than to increase the height; hence spans, whether single or in series, should be wide and low.

For culverts the dimensions should be about as follows:—

Span	3′	4′	5′	6′	8′	10′	12′
Height from water run or floor to underside of keystone	3¼′	4′	4¾′	5½′	7′	8½′	10′
Total area of opening sq. ft.	9′	14′	21′	29′	49′	74′	96′

8. Where the height from the bed of the stream to the surface of the road is limited to the minimum requirements, the general proportions of the span should be based on the average depth of the water at Observed High Flood Level. **Arch Bridges for Minimum Headway or clearance.**

The following proportions are recommended:—

	Span equals.	A equals.	B equals.
Rise of arch = ⅕ of span	4·4 d	2·2 d	1·32 d
,, = ¼ ,,	4·6 d	2·3 d	1·15 d
,, = ⅓ ,,	4·8 d	2·4 d	·80 d
,, = ½ ,,	5·55d	2·77d	0

Where A = height from average bed level to underside of crown of arch.

B = height from average springing of the arch.

d = height from average O. H. F. Level.

i.e., if the waterway is contracted

$$d = \frac{\text{Water area before obstruction}}{\text{Sum of clear span openings.}}$$

These proportions give a total area of arch opening of 1·94 times the waterway area which is sufficient area to pass a flood of 3 times the amount flowing at observed high flood. If necessary the level of the road should be raised to suit spans of not less than the above proportions and clearances, but beam or R. S. Joist bridges are more suitable for such cases.

Calculations for Waterway and Depths of Foundations.

9. Practical experience of bridges and causeways shows the great importance of taking the foundations to a depth where they will be below the erosion caused by the greatest possible flood. **Necessity for calculations for waterway and scour.**

The object of calculating the waterway and scour is to ascertain the amount of waterway necessary, the effect of the waterway provided on the velocity of the stream, and the effect of the resultant velocity on the scour, the depth to which the foundations should be taken being determined therefrom.

The calculation and design of bridges is, however, in particular a branch of the Engineer's art in which theory must be tempered by practice, just as practice must equally be guided by theory and experience. The methods of calculation here indicated, which are based upon a careful study of practical results on the N.-W. Frontier of India (to the conditions applicable to which and similar localities they are primarily applicable), combined with a study of accepted principles in regard to bridge design and of practice in America and elsewhere, should therefore be utilized, to obtain data to be adopted as a safe guide, and to be modified in the light of special local conditions and actual practical experience where necessary.

The calculations indicated should always be made, wherever possible; safe rough rules for concurrent or emergency use are also given.

Drawings for bridges.

10. Drawings for bridges should consist of :-

(i) A site plan, drawn to 1/100" to 1/400" scale.

(ii) Cross sections of the river bed, drawn to 1/20" to 1/10" scale horizontally and 1/10" to 1/20" vertically, showing bed level, observed high flood level, and all other useful information relating to the hydraulics of the river.

(iii) Plans and elevation of the bridge, drawn to 1/16" to 1/40" scale showing centres and width of piers, depth of foundations, bed of river, observed high flood level and clearance to girders. Girders, abutments, and piers to be drawn in outline only.

(iv) Details of each type of pier and abutment, drawn to ⅛" scale.

(*N.B.*—⅛" scale—scale of 8 ft. to 1 inch, and so on.)

These scales are suitable for a bridge about 600 ft. long.

For smaller bridges larger scales may be used, but the drawings should not exceed 24 inches wide by 36 inches long.

The scale of a drawing depends on the size of the smallest dimensions. There is no object in drawing the bridge elevation to ¼" scale necessitating a drawing 12 ft. long, when only the centres of piers are to be shown. The bridge diagrams in this Handbook have been drawn to reduced scales; about 1½ times to twice these scales would be normal.

Siting Bridges.

11. The site of a bridge should be in a straight reach of the river where the current is straight, and the banks are regular, approximately parallel, and well defined. The site should be far enough below large tributaries and bends to be free from their disturbing effect on the current. Where a causeway already exists the bridge should preferably be sited above it, as the causeway will cause the stream to head up and will thus tend to reduce the scour.

Bridges which contract the waterway, or are sited where the banks come close together, must have extra deep foundations and adequate training works, to ensure that the current will flow straight and not diagonally through the bridge.

12. The most reliable method of estimating the discharge or run off from a catchment area is by calculating what the discharge of the river would be at the observed high flood level. Run off.

In the absence of any such local data, any attempt to calculate run off and waterways from first principles is liable to very serious error, as many of the factors can only be determined by reliable data based on extensive observation, and such information will never be available where even flood information cannot be obtained.

The Run Off depends upon :—

(1) The extent, duration and intensity of rainfall on the catchment area.
(2) The usual path of rain storms, whether up or down or across the catchment area.
(3) The natural or artificial storage.
(4) The size and shape of the catchment area and its gradient.
(5) The character of the ground, whether bare and rocky, or absorptive or covered with vegetation.
(6) The time which the precipitated rain takes to reach the site from every part of the catchment area due to the formation of the catchment area, *i.e.*, its slope.

The waterway area depends upon the run off and the velocity of the stream, which depends on the gradient, the hydraulic radius, and the roughness of the bed.

13. For N.-W. Frontier rivers the run off from the catchment area is approximately represented by the following formulæ :— Run off Formulæ.

Q = maximum discharge or run off in cubic feet per second.

M = catchment area in square miles above the site of the Bridge; this may be measured from the Survey of India maps.

For M less than 9½ square miles $Q=1,200\ M^{3/4}$ . . . (1)
For M 9½ to 12,000 square miles $Q=2,100\ M^{1/2}$ . . . (2)
with a variation above or below of 33 per cent. in both cases.

Colonel Dicken's formula for all-India is $Q=CM^{3/4}$. (3)
in which C averages about 825.

Ryve's formula for Madras district is $Q=CM^{2/3}$ . . (4)
in which C = 450 within 15 miles of the coast.
= 563 for 15 to 100 miles from the coast.
= 675 for limited areas near the hills.

These formulæ are to be used as a guide in conjunction with calculations for Q (see para. 18) from the known H. F. L.

The formulæ must not be used mechanically, neglecting local information. Where local information is unreliable the formulæ give an approximate idea of the discharge.

Waterway Table.

14. Where local information is not available, the waterway area, especially for small bridges and culverts, may be approximately determined from the Dun Drainage Table (see Table VIII).

This waterway table is based on reliable flood data in the United States, and is used extensively there with satisfactory results.

From observations of existing culverts and bridges in any district, the particular percentage column of the Table applicable to that district may be ascertained.

As run off formulæ for American and Indian rivers increase at about the same rate for an increase in catchment area, the waterway areas will also increase similarly.

The percentage values indicate the relation of the waterway areas in the four columns, and have nothing to do with the ratio of run off to precipitation.

Use the 120 per cent. column in bare stone covered hills liable to heavy rain storms.

Use the 100 per cent. column in hills covered with vegetation.

Use the 80 per cent. column for plains close to the hills.

Use the 50 per cent. column for plains distant from hills.

These are approximate percentage values, and they may require modification for special conditions, and from local experience with existing culverts and bridges.

Definitions.

15. The Moving Bed is the depth of bed in motion during floods.

The Bed of the Stream is the surface of the moving bed.

The Solid Bed is the bottom of the moving bed.

Existing Scour is the greatest depth of the moving bed before obstruction by the bridge abutments and piers.

Ultimate Scour is the greatest depth of moving bed after obstruction.

Observed High Flood Level is the level of the highest recorded flood.

Maximum High Flood Level is the level of the highest possible flood.

See Plate IX.

Notation.

16. The notation adopted in the formulæ is as follows :—

(All units feet and seconds.)

At Observed High Flood Level (O. H. F. L.).	*At Maximum High Flood Level (Maximum H. F. L.).*
V = average velocity . . .	V_1
A = area of waterway cross-section	A_1

P = wetted perimeter of river bed for A . . . . . . P_1

r = hydraulic mean depth . . $= \frac{A}{P}$ $r_1 = \frac{A_1}{P_1}$

Q = total discharge in cusecs . $Q_1 = 1\frac{1}{3}$ Q (minimum).

s = slope of water surface . . = fall of water surface divided by the length in which it occurs.

n = co-efficient of rugosity of river bed.

= ·020 for earth in good order and regimen free from stones and weeds . . . . . .

= ·025 earth in fair order and regimen free from stones and weeds . . . . . .

= ·030 earth in bad order, occasional stones and weeds . . Kutter.

= ·035 for rivers in bad order and regimen with stones and weeds. (Used for hill streams generally) . . . . . .

= ·050 for torrential rivers in beds covered with detritus and boulders . . . .

At Afflux.

B = clear unobstructed width of waterway between piers and abutments.

A_2 = area of waterway cross-section over width B at Maximum H. F. L.

d = average depth of $A_2 = \frac{A_2}{B}$

c = ·80 for round nosed piers and ·70 for triangular nosed piers.

q = discharge in cusecs per foot of width = $\frac{Q_1}{B}$

H = total velocity head at obstruction before scour.

h = afflux or rise of water surface caused by the obstruction.

V_2 = average velocity at obstruction before scour (under the arches or girders).

A_3 = empiric average waterway area under bridge for calculating V_2, making allowance for afflux and scour.

Scour.

m = Scour co-efficient for the material of which the bed is composed.

= ·63 for very fine silt (as in Sind canals) . . . . . (Buckley).

= ·84 for fine sand silt (as in Punjab canals) . . . . }
= 1·00 for sandy loam . . } (Kennedy).
= 1·07 for coarse silt and coarse sand . . . . }

= 1·2 to 1·5 for sand and small bajri . . . . }
= 2·5 to 3·0 for bajri and gravel . } calculated from data of streams on N.-W. Frontier of India.
= 3·0 to 3·5 for gravel and boulders . . . . }

D = maximum depth to scour holes and channels (in the vicinity) below O. H. F. L. before construction.

D_1 = maximum depth to scour holes and channels (in the vicinity) Maximum H. F. L. after obstruction.

D_2 = depth of foundations below Maximum H. F. L.

V_3 = velocity after obstruction and scour.

Miscellaneous.

C = clearance between O. H. F. L. and lowest part of bridge superstructure.

R. L. = reduced level referred to datum.

Data required from bridge site.

17. Take 3 points (*a*),(*b*), (*c*) on the river under consideration :—

(*a*) at a distance above the bridge site equal to 6 times its width between banks,

(*b*) at the bridge site,

(*c*) at a distance equal to (*a*) below the bridge site.

If appreciable tributaries or waterfalls occur between (*a*) and (*c*), the distance (*a*) to (*b*) and (*b*) to (*c*) should be altered to avoid them, or the calculations modified to allow for them.

At each of these points plot a cross-section at right angles to the stream, showing with reference to one datum level :—

(i) the water level,

(ii) the observed high flood level by enquiry and inspection of the banks,

(iii) the bed of the stream,

(iv) the maximum depth of scour channels in the vicinity, immediately after a flood if possible or ascertained by trial pits.

Measure and record the distances between (*a*), (*b*) and (*c*), and with a surveyor's level ascertain very accurately the fall of the water surface or O. H. F. L. between them on the same day.

Note the character of the bed and banks, and if the current is straight or winding. Also. (with a view to determining the co-efficient of rugosity n), note the material composing the bed of the river, *e.g.*, size of boulders, gravel or sand, etc., and by one or two trial pits ascertain if the material underneath the bed is coarser or finer than at the surface.

In respect of each of these three cross-sections,

(i) Calculate and record the cross-sectional area of the waterway A at O. H. F.

(ii) Measure and record the wetted perimeter P at O. H. F.

(iii) Calculate and record the hydraulic mean depth below O. H. F. L., $r = \frac{A}{P}$

(iv) Measure and record the maximum depth to scour holes, D, below O. H. F. L.

(v) Calculate and record the slope of the water surface, $s = \frac{\text{fall}}{\text{distance}}$

18. To find the mean velocity at O. H. F., Manning's formula should be used — Mean Velocity and Discharge at observed high flood.

$$V = \frac{1{\cdot}486}{n} \times r^{\frac{2}{3}} \times s^{\frac{1}{2}} \quad . \quad . \quad . \quad . \quad . \quad (1)$$

The values for the co-efficient of rugosity of the river bed, n, are given in paragraph 16. These are the values given in Kutter's formula; they are equally applicable to Manning's formula, which gives approximately the same results as Kutter's, and is easier to use.

If the waterway includes several channels of greatly varying depths, each channel should be calculated for separately.

Tabulated $\frac{2}{3}$ roots and square roots are given in Table X.

The discharge at O. H. F.

$$Q = A. V. \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

The values of V and Q should be calculated by equations (1) and (2), and recorded, for the three sites.

Assuming that an appreciable waterway or waterfall does not occur between sites (*a*) and (*c*), Q for the bridge site (*b*) should equal the mean value of Q for sites (*a*) and (*c*), and unless the value of Q for the bridge site (*b*) differs by more than 5 per cent. from the mean value for sites (*a*) and (*c*), the calculation should proceed utilizing the data for the bridge site.

If the value of Q for the bridge site differs by more than 5 per cent. from the mean value for sites (*a*) and (*c*), an error needing rectification in data or calculations is indicated, and they should be retaken.

If the resultant value of Q for the bridge site then still differs by more than 5 per cent. from the average for sites (*a*) and (*c*), the mean value of Q for the three sites nust be adopted, and

$$V \text{ taken} = \frac{\text{mean value of } Q}{A \text{ for bridge site}}$$

(the remaining data for the bridge site of course being used in any case).

In a case in which an appreciable tributary or waterfall occurs between sites (*a*) and (*c*), its discharge must be calculated, and allowed for in comparing Q for (*a*), (*b*), and (*c*).

Velocity and Discharge at maximum high flood.

19. Measure from the map the catchment area M in square miles discharging through the bridge.

Vide paragraph 13, the maximum discharge or run off

$$Q_1 = 1{,}200\ M^{3/4} \text{ cusecs where M is less than } 9\tfrac{1}{2} \text{ sq. m.} \quad (3a)$$

$$\text{or } Q_1 = 2{,}100\ M^{2/3} \text{ cusecs where M is } 9\tfrac{1}{2} \text{ sq. m. or over} \quad (3b)$$

subject to a variation of 33 per cent. in special cases.

To allow for an abnormal flood, the discharge at Maximum H. F., Q_1, must be taken as a suitable multiple of Q, approximately $= Q_1$, calculated by the run off formula, subject to a minimum value

$$Q_1 = 1\tfrac{1}{3}\ Q \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

In any case, Q_1 should not ordinarily exceed 2Q.

From equation (1) it may be proved that if the rise in water surface does not increase the wetted perimeter by more than 50 per cent. the hydraulic mean depth at Maximum H. F.

$$r_1 = \left(\frac{Q_1}{Q}\right)^{3/5} \times r \text{ approximate} \quad . \quad . \quad . \quad . \quad (5)$$

The approximate rise in surface at

$$\text{Maximum H. F.} = r_1 - r \quad . \quad . \quad . \quad . \quad . \quad (6)$$

The wetted perimeter at Maximum H. F.

$$P_1 = P + 2\,(r_1 - r) \text{ approximate} \quad . \quad . \quad . \quad (7)$$

The waterway area at Maximum H. F.

$$A_1 = P_1 \times r_1 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (8)$$

From equation (7), the probable maximum velocity

$$V_1 = \frac{1{\cdot}486}{n} \times r_1^{2/3} \times s^{1/2} \quad . \quad . \quad . \quad . \quad . \quad . \quad (9a)$$

$$= V \times \left(\frac{r_1}{r}\right)^{2/3} \quad (9b)$$

$$= V \times \left(\frac{Q_1}{Q}\right)^{2/5} \quad . \quad . \quad . \quad . \quad . \quad . \quad (9c)$$

The maximum discharge

$$Q_1 = A_1 \times V_1 \quad . \quad . \quad . \quad . \quad . \quad . \quad (10)$$

This value should be approximately = the value assumed for Q_1 after calculating the probable maximum run off, the observations and calculations thus being counter-checked. As noted in paragraph 13, the run off formulæ must not be used mechanically, or local information neglected. In case of doubt higher rather than lower alternative values should be adopted.

The R. L. of Maximum H. F. without afflux = R. L. of O. H. F. $+(r_1 - r)$.

20. A suitable method of calculating the afflux, or rise above calculated Maximum H. F. L. due to an obstruction, such as that caused by the bridge piers, etc., is given below. Afflux.

Plot on the cross-section the proposed bridge spans and piers, and calculate from the cross-section the Area A_2 of the clear waterway between piers and abutments at Maximum H. F.

The average depth

$$d = \frac{A_2}{B} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (11)$$

where B = the total length of clear waterway between piers.

The average discharge per ft. width of clear opening

$$q = \frac{Q_1}{B} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (12)$$

The afflux should be calculated by the use of Merriman's Formula (given in Merriman's Hydraulics).

$$H^{\frac{1}{2}} (H + 1\tfrac{1}{2}\, d) = \frac{q}{5{\cdot}35\, c} \quad . \quad . \quad . \quad . \quad . \quad . \quad (13)$$

whence the afflux

$$h = H - {\cdot}0155\, V_1^2 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (14)$$

H is the total velocity head at obstruction before scour, and c is a constant depending on the shape of the piers (for values see paragraph 16).

Equation (13) is solved, to find H, by trial and error, utilizing the table of powers given on Plate X.

The effect of scour on the afflux is not allowed for in this formula; hence after scour the afflux will be less than that calculated.

21. In calculating the velocity through the bridge, 10 per cent. should be added to allow for the effect on the velocity of the contraction of the current past the piers and abutments. Velocity and obstruction before scour.

$$\text{Hence } V_2 = \frac{1{\cdot}1 \times A_1 V_1}{A_3} \quad . \quad . \quad . \quad . \quad . \quad . \quad (15)$$

$$\text{Where } A_3 = \frac{d + (d + h)}{2} \times B = (d + \frac{h}{2})B \quad . \quad . \quad (16)$$

22. On account of the obstruction caused by the piers and abutments, and the increased velocity due thereto, the depth of scour Scour.

will be increased at the bridge. The increase in depth of scour will bear a relation to the increase in velocity.

The following considerations of the effect of velocity on scour in beds of different materials are of general interest. The results thereby obtained are not directly used in the formulæ for bridge calculations here given; the results obtained by the latter are, however, in conformity with these principles, and the velocity-material data will be found to be of particular use in designing training bunds (see Chapter VIII).

Velocity of water to move stones.

23. By Chailly's formula, the velocity of water V in ft. per sec. which will move stones of diameter d ft. and specific gravity G, is

$$V = 5{\cdot}67\sqrt{G.\,d} \qquad . \quad . \quad . \quad . \quad . \quad . \quad . \qquad (17)$$

For practical purposes, assume $G = 2{\cdot}65$ (value for sand and gravel),

$$\text{Then } d = \frac{V^2}{85} \qquad . \quad . \quad . \quad . \quad . \qquad (18)$$

From this the following data result:—

A velocity of ft. sec .	1/2	1	2	3	4	5	7	10	15	20
Will move stones of diameter	1/28″	1/7″	5/8″	1¼″	2¼″	3½″	7′	1′ 2″	2′ 8″	4′ 8″

Critical velocity and co-efficient of scour.

By Kennedy's formula the critical velocity which causes neither silting nor scouring

$$V_0 = m\ D^{\cdot 64} \qquad . \quad . \quad . \quad . \quad . \quad . \quad . \qquad (19)$$

Whence

$$D = \left(\frac{V_0}{m}\right)^{1\cdot 56} \qquad (20)$$

and

$$m = \frac{V_0}{D^{\cdot 64}} \qquad (21)$$

D being the maximum depth, and m a coefficient depending on the composition of the bed

If the velocity be greater or less than V_0, the bed will scour or silt up until a stable condition is attached, as expressed by the above formula.

By inserting in equation (20) the observed or calculated value of V, and the corresponding value of D from water level to the bottom of the deepest scour hole or channel, during flood conditions, the value of m can be calculated in any individual case.

The values of m so obtained will vary slightly for the same material, according to local currents and conditions, which may cause a comparatively large or small scour; such variations are useful.

The usual values for m are given in paragraph 16. The values for sand and bajri, bajri and gravel, and gravel and boulders have

been calculated from observation of twelve typical streams on the Indian N.-W. Frontier, by equation (21).

24. If it be assumed that the bed will scour uniformly, the discharge at the deepest part of the bed after scour will be the same as before scour. The resulting equation will give the minimum scour, which is the most favourable condition. **First empiric method of calculating scour.**

If V_3 be the velocity after scour

$$D\ V_2 = D_1\ V_3$$

and from equation (19), $V_3 = m\ D^{\cdot 64}$

Hence the minimum depth of scour below water level (neglecting the afflux)

$$D_1 = \left(\frac{D\ V_2}{m}\right)^{\cdot 61} \qquad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \qquad (22)$$

Or, substituting for m $\frac{V_0}{D^{\cdot 64}}$ (equation 21) and thus cancelling out m

$$D_1 = D\left(\frac{V_2}{V_0}\right)^{\cdot 61} \qquad . \quad . \quad . \quad . \quad . \quad . \quad . \qquad (23)$$

If there be no limit to the discharge at the deepest part of the bed, and the discharge increase with the scour so as to maintain the velocity V_2, from equation (20) the maximum depth of scour below water level (neglecting the afflux) **Second empiric method of calculating scour.**

$$D_1 = D \times \frac{\left(\frac{V^2}{m}\right)^{1\cdot 56}}{\left(\frac{V_0}{m}\right)^{1\cdot 5}}$$

$$= D\left(\frac{V_2}{V_0}\right)^{1\cdot 56} \qquad . \quad . \quad . \quad . \quad . \qquad (24)$$

The first method is suitable for channels of uniform cross-section such as canals, in which the velocity is uniform over the cross-section, and in which any increasein velocity would normally cause an uniform scour over the whole bed.

The second method is suitable for rivers and streams where the section is not uniform and where there are main currents which scour channels in the bed.

For the small increase in velocity due to a bridge the second method is the more accurate, but for large increases in velocity the scour is more uniformly distributed over the section and the conditions approach those suitable to the first method.

The practical method given below, which is a compromise between the above two methods, is suitable for general use in bridge calculations, and gives results in agreement with collated actual scour records. **Practical method of calculating scour.**

The equations are suitable for values of V_2 not exceeding 2 V.

The maximum probable depth of scour below Maximum H. F. L. (neglecting the afflux).

(i) For average sites, waterway slightly contracted, current straight

$$D_1 = \frac{1 \cdot 3\, D\, V_2}{V} \text{ or } \frac{2 \cdot 1\, r\, V_2}{V} \quad \ldots \ldots \quad (25a)$$

(ii) For bad sites, waterway contracted, diagonal currents

$$D_1 = \frac{1 \cdot 5\, D\, V_2}{V} \text{ or } \frac{2 \cdot 9\, r\, V_2}{V} \quad \ldots \ldots \quad (25b)$$

The larger value for D_1 to be adopted in each case.

Depth of Foundations.

25. To allow for local scour and to provide a factor of safety, the foundations should be taken to a depth below Maximum H. F. L. $\frac{1}{3}$ greater than the calculated depth of maximum scour below Maximum H. F. L.

Thus the depth of foundations below Maximum H. F. L.

$$D_2 = 1\tfrac{1}{3} \times D_1 \quad \ldots \ldots \ldots \quad (26)$$

The depth below average bed level

$$= 1\tfrac{1}{3}\, D_1 - d \quad \ldots \ldots \ldots \quad (27)$$

The foundations should be taken down to the depths so calculated, unless rock or inerodible material is met at a lesser depth. In estimating, the calculated depth as above should be allowed for.

As a general principle, the depth of the foundations must be such that they go well down into inerodible material, which is reached (failing rock being met) when the boulders met with are of a size definitely larger than that usually met in the bed of the stream, and the depths arrived at by theory must be applied accordingly in practice.

Approximate rules for depths of foundations.

26. As the depths to scour before and after obstruction are proportional to the velocities after and before obstruction (see equations), the depths to scour and to foundation can be roughly approximated as follows, where there are no data except the values of D and r at O. H. F.

$$V_2 = \frac{1 \cdot 1 \times A_1 V_1}{A_3} \text{ (equation 15)}$$

$= \text{approximately } 1 \cdot 1 \times V_1$ (disregarding the difference between A_1 and A_3)

$$= 1 \cdot 1 \times \left(\frac{Q_1}{Q}\right)^{\frac{2}{5}} \times V \text{ (equation 9c)}$$

$$= 1 \cdot 1 \times \left(\tfrac{5}{3}\right)^{\frac{2}{5}} \times V \text{ (for a maximum discharge } Q_1 = \frac{2\,Q + 1\tfrac{1}{3}\,Q}{2} = 1\tfrac{2}{3}\,Q)$$

$$= 1 \cdot 35\, V$$

Inserting this in equations (25a) and (25b), the following approximate rules result :—

(i) For average sites, waterway not contracted, current straight

Depth to scour below average bed level = 1·0 D or 1·6 r . . . . (28a)

Depth to foundations below average bed level = 1·6 D or 2·5 r . . . . (29a)

(ii) For bad sites, waterway contracted, diagonal currents

Depth to scour below average bed level = 1·4 D or 2·7 r . . . . (28b)

Depth to foundations below average bed level = 2·1 D or 4·0 r . . . . (29b)

The larger of the two values to be used in all cases.

These rules give results within about 5 per cent. of the calculated results, in normal cases.

See Example 1, Chapter VII.

27. The clearance between the superstructure and O. H. F. L. should be sufficient to pass exceptional floods and to allow for some of the waterway silting up. Clearance.

The clearance required above O. H. F. L.

$$C = \left(\frac{Q_1}{Q}\right)^{\frac{3}{5}} r - r \quad . \quad . \quad . \quad . \quad . \quad . \quad (21)$$

To allow for abnormal floods, etc., the minimum values for C should as a rule be not less than the following :—

r	*C*
0 to 12 ft.	¾ r
12 to 18 ft.	9 ft.
18 ft. or over	½ r

Subject to the above minimum values for C, the height of the piers and the level of the superstructure are of course decided by the governing levels of the road alignment on the banks, and in the case of a navigable stream additional headway must be given for traffic.

Calculations for load on Bridges and Culverts on Mechanical Transport and Cart Roads.

28. The live loads for design should be based on the maximum probable loading in relation to the span, and not on the maximum possible. For exceptional cases it is permissible to increase the stresses by 25 per cent. or 30 per cent. As the span increases the probability of heavy loading over the whole span decreases. Live Loads.

29. Impact varies with the span, and with the speed of traffic and the method of propulsion and is expressed approximately by the following formula, in which $x = 1$ for steam railroads, ½ for Impact.

electric cars, ⅓ for motor lorries, etc., and ¼ for road rollers, crowds and cattle.

n = number of railway tracks, or

$\frac{\text{width of road in ft.}}{20}$ – – for roads.

L = loaded length in feet.

$$\text{Impact load} = \text{live load} \times \frac{300}{nL + 300}$$

At speeds less than 5 to 10 miles per hour there is practically no impact, hence when dealing with exceptional loads due to crowded vehicles, people, or animals, no impact need be allowed.

For simplicity all the loads here given for design include an impact allowance, and no further allowance need be made.

Load to be allowed for in floor system.

30. The decking or flooring system should be designed to allow for one 12 ton road roller plus 25 per cent. impact, with the following dimensions and weights:

Wheel base (distance between centres of axles)	10 ft	
Gauge (distance between rear wheels)	5 ft	
Width of tyres . . .	1·32 ft.	
Load (including 25 per cent. impact) on front axle . . .	6·87 tons	– 15,400 lbs.
Load (including 25 per cent. impact) on rear axle . . .	10 tons	— 22,400 lbs.

Distribution of Concentrated Loads.

31. The following rules for the distribution of concentrated loads in calculating stresses in slabs, stringers, and floor beams are based on tests made in the United States (see "Design of Highway Bridges" by Ketchum).

These rules may be applied to the design of bridge floors composed of reinforced concrete slabs, jack arches, or steel troughing or similar construction.

e = effective width of slab in feet (at right angles to its span) which resists the stresses due to the concentrated load. (Extra transverse reinforcement is not necessary to ensure this distribution).

e must not exceed ⅔ of the width of the slab.

L = span of slab in ft. centre to centre of bearings.

c = width of tyre — 1·32′ for a 12 ton Roller.

For caterpillars c = width of track for longitudinal slabs, and = length plus width of track for cross slabs.

Loads on longitudinal slabs (*i.e.*, in the direction of the length of the road) are taken as concentrated, caterpillar loads as distributed along the track, and on cross slabs (*i.e.*, across the road)

both are taken as distributed over the width of the tyre or track in the direction of the slab span in all cases.

The moment per ft. width of slab $= \dfrac{M}{e}$

For formulæ for calculating bending moments M, see Plate XI.

Longitudinal Slabs.

See Plate XI and Table XII.

For bending moment $e = \frac{2}{3}(L + c)$, with a maximum for e of 5 ft. (gauge) for half the rear axle and half the front axle loads; for the whole road roller maximum e = 10 ft.

For shear, for half axle loads e = 5 ft.; for whole axle loads e = 10 ft.

Cross Slabs.

See Plate XI and Table XIII.

For bending moment $e = \frac{2}{3}(L + c)$, with a maximum for e of 10 ft. (wheel base) for one axle load.

For shear e is the same as for moment, with a minimum of 5 ft., and a maximum of 10 ft. (wheel base) for one axle load.

Longitudinal beam or Stringer.

See Plate XI and Table XII.

The axle loads are distributed on a line 10 ft. long (2 × gauge) along the axle.

Hence for a stringer spacing up to 10 ft., the bending moment on one stringer

$$= \frac{(\text{Bending moment from Roller}) \times (\text{Spacing of stringers in ft.})}{10 \text{ ft.}}$$

Outside stringers should be designed for the same moments as intermediate stringers.

Floor Beams.

See Plate XI and Table XIII.

Axle loads are distributed on a line 10 ft long (2 × gauge) along the axle, and 5 ft. long at right angles to the axle.

Hence for floor beams at 5 ft. apart or less, the distributed load per foot of span

$$= \frac{(\text{Maximum axle load})}{10 \text{ ft.}} \times \frac{(\text{Spacing of floor beams in ft.})}{5 \text{ ft.}}$$

and for floor beams more than 5 ft. apart the distributed load per

$$\text{foot of span} = \frac{\text{Maximum reaction on floor beam}}{10 \text{ ft.}}$$

Timber Floors.	Thickness of Floor for 12 ton road roller.								
Spacing of stringers on groups of stringers.	12"	15"	18"	21"	24"	27"	30"	33"	36"
Thickness for 12" Planks . .	2"	2½'	3"	3⅜	3¾"	4"	4⅜"	4⅝'	4⅞"
Thickness for 8" Planks . .	2½"	3"	3¾"	4¼	4⅝"	5"	5⅜"	5½"	6"

Stringers for Timber Flooring.

Axle loads are distributed on a line 8 ft. long (1·6 × gauge) along the axle.

Hence the bending moment on one stringer

$$= \frac{\text{(Bending moment from Roller)} \times \text{(Spacing of stringers in ft.)}}{8\text{ ft.}}$$

If the stringers are in groups, the spacing is the distance centre to centre of each group, and the above is the moment on each group.

Girders, Trusses, and Arches.

Concentrated loads are fairly well distributed in girders and trusses, and in arch ribs or rings by the rigidity of the floor system in the case of open spandrel arches, or by the earth filling in the case of filled spandrel arches. Hence an equivalent uniform load of w lbs. per square foot of road surface may be used without appreciable error, to simplify design.

Up to 50 ft. span $w = \frac{\text{(Moment for Roller)}}{\text{Span}^2 \times 10} \times 8$ lbs. square feet.

Over 50 ft. span w gradually decreases to 70 lbs. sq. ft.

The following uniform loads per square foot of road surface should be allowed for in the design of Girders, Trusses, and Arch Rings or Ribs. (These loads include impact allowance.)

Span.	Girders and Trusses.	Arch Rings or Ribs	Span.	Girders and Trusses.	Arch Rings or Ribs.	Span.	Girders and Trusses.	Arch Rings or Ribs.
ft.	lbs./sq. ft.	lbs./sq. ft	ft.	lbs./sq. ft.	lbs./sq.ft.	ft.	lbs./sq. ft.	lbs./sq.ft.
10	450	450	45	140	190	120	95	130
15	300	420	50	130	180	140	90	125
20	240	340	60	120	170	160	85	120
25	210	300	70	115	160	180	80	110
30	190	270	80	110	155	200	75	105
35	170	240	90	105	145	over 200	70	100
40	150	210	100	100	140	..	..	..

For intermediate spans loads should be proportionately interpolated. The loads given for arches are about 40 per cent. greater than for Trusses, as partial loading gives the maximum stresses.

Explanation of Tables.

32. Plate XI gives the loading for a 12-ton road roller, as described above, and the methods of calculating maximum moments and reactions.

Tables XII, XIII and XIV give tabulated results, for practical use, of the moments in longitudinal and cross slabs, stringers, and longitudinal girders, and of the reactions on piers and floor beams, together with particulars of corresponding calculated thicknesses of reinforced concrete slab decking, and dimensions and spacing of reinforcement bars.

Table XII.

Column 2 is calculated according to the formulae on Plate XI.

Columns 3 and 4 are obvious, *i.e.*, column 2 divided by 2 and 10, respectively.

Columns 5 and 6 are the section modulus $Z=I/y$ per foot of width between centres of stringers sufficient to take the roller and the dead load.

For stringers, *i.e.*, longitudinal rolled steel joists at 3 ft. centres multiply by 3 ; at 4 ft. centres multiply by 4.

Column 7 gives effective width of slab to carry a concentrated load.

Column 8 is column 3 divided by column 7.

Columns 9 to 12 are the total thickness of slab necessary to take the roller and the dead load.

Example.

18′ span centre to centre of bearings, rolled steel joists 3 ft. apart.

Wearing surface 70 lbs. /sq ft.

Z required$=11{\cdot}61\times3=34{\cdot}83$.

12″×5″ @ 32 lbs rolled steel joists, for which $Z=36{\cdot}66$, will do.

Table XIII.

Column 2 is calculated according to the formulae on Plate XI.

Column 3—effective width of slab to carry a concentrated load is 2/3 (span *plus* width of tyre 1·32′), maximum 10 ft.

Column 10 is the maximum reaction for live load for various spans calculated according to the formulae on Plate XI.

Column 12 is the live load per foot of floor beam (transverse or cross girder at right angles to centre line of road) span for various spacings of floor beams.

Table XIV.

Example.—12′ clear span between piers metalled, span centre to centre of bearings about 13 feet. From Table XII, column 11, a 12¾″ slab is required, and from Table XIV for this thickness use ¾″ bars at 5¾″ centres placed 1½″ above the underside of the slab.

If there were a series of continuous spans, the end spans would be half continuous and the average of columns 11 and 12, Table XII, is 11⅝″ say a 11¾″ slab. From Table XIV use ¾″ bars at 6¼″ centres for 11⅝″ thickness.

Plate XXXV gives the position of the bends in the bars.

Wind Loads for Trusses.

33. Wind loads on trusses should be taken as follows :—

(i) A horizontal load on the loaded chord of 300 lbs. per lineal foot.

(ii) A horizontal load on the unloaded chord of 150 lbs. per lineal foot.

Both should be treated as moving loads, and no impact should be added.

Capacity of bridges so designed.

34. Floor systems, girders, trusses, and arches designed for these concentrated and uniform loads will safely carry the traffic detailed below.

These particulars are based upon tables given in the "Memorandum on Road Bridges" published by the War Office in 1918. Reference should be made to these or similar later official tables in regard to other vehicles not mentioned hereunder, and the list should be kept up to date.

Normal.

Two 12-ton road rollers abreast, or in file at 30 ft. clear intervals.

15 ton caterpiller tractors at 30 ft. clear intervals.

A double line of 3-ton motor lorries at a check. (8 tons loaded).

All marching formations of cavalry, field artillery, infantry, camel, mule, and army transport, passing similar formations on the bridge.

All crowds of pedestrians and cattle, pack animals, and carts.

Any train of loads in which no axle load exceed 8 tons nor caterpiller axle load 12 tons, and in which the load per sq. ft.

$$w = \frac{\text{total load in lbs.} \times 1{\cdot}25 \text{ for impact}}{10' \times (\text{length of train} + 10')}$$

does not exceed that given in the table for a span equal to the length of the train + 10′.

Emergency.

Bridges designed as above will also take the following loads at safe stresses with restrictions, *i.e.*, speed 4 miles per hour, load on centre of road, and no other load alongside :—

18 ton Whippet Tanks at 25 ft. clear intervals.

16-ton Wheel Tractors at 25 ft. clear intervals.

8″ Howitzer hauled by 11-ton caterpillar, subject to the floor stress being increased by 5 per cent.

6″ B. L. Gun, Mark VII, hauled by 16-ton Tractor, subject to the floor stress being increased by 25 per cent.

12″ Howitzer hauled in two trains by 14-ton caterpillars, trains at 60′ clear interval.

9·2″ Howitzer hauled by 11 ton caterpillar.

6″ Gun Mark XIX hauled by 11-ton caterpillar.

Any train of loads in which no axle load exceeds 12 tons nor caterpillar axle load 18 tons, and in which the load per square foot

$$w = \frac{\text{total load in lbs.}}{\text{width of road} \times (\text{length of train} + 10')}$$

does not exceed that given in the table for a span equal to the length of the train + 10′.

N.B.—These bridges will *not* take 30-ton Tanks.

Calculations for Load on Bridges and Culverts on Pack Transport Roads.

35. Bridges and culverts on pack transport roads should be calculated to take equestrians, pedestrians, and pack animals (or cavalry, infantry, and transport mules and camels), crowded

36. The maximum concentrated load from cavalry in marching order is about 850 lbs. on the fore feet, which on a 3 ft. width × 4 ft between fore and hind legs gives an equivalent uniform distribution load for bending moment of 110 lbs /sq. ft. Transport Mules and Camels give smaller concentrated loads. Infantry in four at a check give a load of about 100 lbs / sq. ft. on an 8 ft. width **Loads to be allowed for.**

As it would be wasteful to design throughout for 110 lbs. per square foot, which would not be in accordance with probabilities, the load should be reduced to 50 lbs. per sq. ft for 100 ft. spans and over.

Also as the above maximum loads are only possible when the traffic is crowded and stationary, no allowance need be made for impact.

For the design of all floors, beams, girders, and trusses the following live loads should be taken :—

Span in feet	1	5	10	15	20	25	30	35	40	45	50	60	80	100 and over
Load in lbs /sq ft. .	150	140	130	120	110	100	90	80	75	70	65	60	55	50

For intermediate spans proportionate loads should be interpolated.

37. If in a bridge the spans of the various parts were as follows :— **Example.**

Floor slabs 5 ft. span, Stringers or floorbeams 10′ span, and Span of girders or trusses 60 ft.

these members would be designed respectively for 140, 130, and 60 lbs. respectively, per square foot of the roadway area carried by them.

CHAPTER VI.

Bridges and culverts—types and constructional details.

Introductory—Type Designs and Proportions for Culverts, Arches, Abutments, and Piers—Normal Types of Flat Bridge Spans up to 50'—Trestle Pile Bridges and Overflow Bridges—Plate Girder and Truss Bridges—Reinforced Concrete Arch Bridges—Reinforced Concrete Bowstring Bridges—Suspension Bridges—Light Bridges on Pack Transport Roads—Boat Bridges—Bridge Defences—Miscellaneous Bridge Details and Specifications—Open Bridge Foundations—Well Foundations and Well Sinking—Piles and Pile Driving.

INTRODUCTORY.

Type designs.

1. Type designs, affording all information necessary for the preparation of working drawings, are given in the appended Plates. These designs must be thoroughly studied to ensure that every point and detail is attended to.

Rough comparative estimates of cost to ascertain which type is the best, having regard to economy combined with practical considerations, can, in most cases, be made direct from these designs.

Details are given as a rule for 18', 15', and 10' roadways.

TYPE DESIGNS AND PROPORTIONS FOR CULVERTS, ARCHES, ABUTMENTS, AND PIERS.

General principles of Culvert design.

2. General constructional details for culverts (distance between abutments not exceeding 12 feet) are shown on Plate XV.

The principles of the use of culverts are given in the General Specifications (Chapter II).

Culverts are spans in which the distance between faces of abutments does not exceed 12 feet.

For economy culverts should ordinarily never consist of more than one span. It is never cheaper to make 2 spans of a culvert except in the case of pipe culverts.

The gradient of culverts should be as great as possible.

Wells at the inlet and outlet may be necessary in hill sections.

Culverts may be of the following types:—Corrugated iron, cast iron, concrete, or reinforced concrete pipes, or arched, stone slab, or reinforced concrete slab.

Pipe or slab culverts will usually be the cheapest for spans under 6 feet. In pipe culverts, which should extend across the formation width, a straight faced wall can generally be given instead of wings, and sometimes wings may be omitted in any case.

Culvert opening should be low and wide; this shape reduces the scour and also the cost of the end wing walls, and prevents undesirable humps in the road, which must be avoided.

Generally it is cheaper to pave the floor in inverted arch form than to take the foundations below scour.

End cut off walls should be provided, and where a culvert is located in an erodible bank, certain walls may also be necessary to prevent the water outflanking the culvert.

3. Proportions of arch bridge and culvert spans are given in Plate XVI. **Proportions of Arch Bridges and Culverts.**

These are suitable for spans from 6 feet upwards. The arrangement of abutments and wing walls is given in Plate XVII.

The proportions of wing walls are explained in Chapter IV (Retaining Walls) and shown in Plate VI.

The economic span for a series of arched spans – 2 H., where H = the distance between the foundations and the crown of the arch. (See Chapter V.)

The shape of the piers in arch bridges and culverts should conform to the shapes shown in Plate XVIII, subject to the thickness necessary for arches shown in Plate XVI.

The equations given for finding the proportions of arches and their abutments are Trautwine's rules (see Trautwine's Civil Engineer's Pocket Book) for spans of 2 feet to 150 feet.

These arches are safe for all military loads. The abutments can safely carry the arch ring before being backfilled, but it is preferable to backfill up to the springing before striking the centres.

The Table on Plate XVI gives the main dimensions for 2 ft. to 50 ft. spans, including the approximate load on the foundations per foot width of arch ring, for a road surface 1 ft. 6 in. above the crown of the arch and for Height = Span. If the fill is more, the excess should be added to the table values. If the height is less than the span the excess weight of the masonry should be deducted (see example 8, Chapter VII).

Abutment type piers should be given at every 5th pier, and arranged symmetrically about the centre of the bridge. They should have a width at the springing of E (see Abutment Formula, Plate XVI), and be battered on both sides at 1 in n.

The thickness of a masonry spandrel wall at its base should be equal to half its depth below the road surface.

It is essential that the foundations of arch bridges shall be indubitably rigid and sound (see also para. 23).

For the special economic rules for arch spans, see Chapter V, paras. 7 and 8.

An example of a masonry arch bridge is given in Chapter VII (Example 8).

Reinforced concrete Arch Bridges. The theory and design of reinforced concrete arch bridges are fully dealt with in paras. 22 to 37, the method described wherein is also applicable with suitable modifications, to masonry and unreinforced concrete arches.

Proportions of Abutments for flat deck bridges and culverts. 4. Proportions of abutments (except for arch spans) are shown in Plate XVII.

Proportions of wing walls are shown in Plate VI.

The dimensions of the base and toe are given as a proportion of the height. At ground level the toe should be projected and the heel continued vertically as shown. This increases the stability of the abutment and reduces the pressure on the foundation at the toe. The heel should not be projected as this increases the toe pressures. On bad foundation material the toe should be projected further and the toe pressures worked out as in the example in Chapter IV. The heel should never be projected as this increases the toe pressures.

In types I and II (see Plate XVII) the spaces between beams are filled with masonry to hold the filling behind the abutment; in types III and IV a small wall on the top of the truss seat is provided for this purpose.

Arrangement of Abutments and Wing Walls for all types. 5. The arrangement of abutments and wing walls for all types of spans is shown in Plate XVII.

In bridges and culverts subject to floods 45° wing walls should be used. Returned wing walls may be used where the bank is above flood water level, but it is cheaper to stop the wall at the top of the bank and allow the fill to spill around the abutment.

Drawings of abutments and wing walls. 6. In the design of abutments and wing walls, the general layout of which is given on Plate XVII, the following points must be attended to.

Having schemed out the sections of the abutment and the wing walls, the abutment and wing walls must be designed so that they will fit simply together. For this purpose it is best to take a clean dividing line parallel to the centre line of the roadway, at a distance from it of half the overall width of the bridge, or half the overall outside width of the masonry parapets, depending on the general arrangement. If in doubt both arrangements should be drawn out, to see which is better. The wing wall face batter (as projected on the above dividing line) should then be drawn on the abutment section, making this batter line intersect the abutment face near the ground level, or at the face of the ballast wall and bridge seat, whichever is more suitable. Below the ground level the faces of both abutment and wing wall must coin-

cide, and the sections must be modified accordingly. This determines the faces of both abutments and wing walls, and the plan should be drawn out accordingly.

The general scheme as shown on Plate XVII should be used as a guide. It is not possible to give every individual case which may occur in practice, but the above rules are correct for all cases.

A scale of $\frac{1}{8}$ inch=1 foot should be used for truss bridge abutments, as drawings on larger scales are of unworkable size.

7. Proportions for piers for flat deck bridge and culvert spans are shown in Plate XVIII. The proportions are based on the height of the piers. Three types are shown, *viz.*:— Piers.

Type I :— For slab, rolled steel joist, and reinforced concrete beam spans, up to 40 or 50 feet.

Types II and III :— For truss spans, and for all spans over 40 or 50 feet.

All three types have the same stability.

In all cases the top thickness t must not be less than the value indicated, which is a minimum. In high battered piers t will depend upon the minimum proportions given in the diagrams for the thickness above the foundations T.

The cut waters shown give the minimum resistance to the stream, combined with economy of material and simplicity in construction. The cut water shape must be carried down to the base. Half round or 45° triangular ends are not much better than square ends, and give excessive scour along the sides and beyond the tail of the pier.

The alternative design I—A is very economical for small spans of troughing or of reinforced concrete slabs, giving a saving of 40 per cent. in the pier masonry.

As piers of cement concrete or cement masonry will be of less thickness, they may be more economical than lime masonry piers, where materials and labour are expensive. In all except small bridges, and including particularly bridges across perennial streams, the specification for piers and abutments should ordinarily be a cement concrete core with stone (or brick) masonry in cement skin up to Nala bed level, and brick or stone masonry in cement above.

For spans under 50 ft. reinforced concrete piers may be used, of $\frac{2}{3}$ the thickness of concrete piers and reinforced with 25 per cent. of vertical steel in each vertical face.

8. The top thickness of piers and abutments depends finally on the size of the superstructure bearings. In the case of truss bridges, the minimum thickness given on Plate XVIII will be exceeded in certain cases, according to the size of the bearings of the trusses to be used, to suit which the piers and abutments must be designed. Pier and Abutment Superstructure Seatings.

Truss bearings should be compactly designed to take the appropriate working stresses in the truss members concerned, consistently with distributing the load over a sufficient area on the piers and abutments, and with minimizing the resultant width of the piers as much as possible, in order to save expense in masonry and foundations, and to avoid unnecessary waterway restriction.

When old railway bridge girders, which may have extravagantly large bearings, are used (as is frequently the practice on Indian N. W. Frontier bridges), the above mentioned desiderata are not always complied with, and unduly wide piers result. In such instances the girder bearings should be remodelled and reduced whenever economically possible.

In efficiently designed road bridge truss bearings the bed plates should be approximately square in plan, with the shorter dimension in the direction of the span.

The clearance between trusses should be about 3½ inches per 100 feet length of span, with a minimum of 4 inches for 80 foot spans. The distance from the edge of the bearing to the size of the pier or abutment should be about 3 inches for 100 foot spans.

The coping or seating on abutments and piers must be of first class work in 1:2·4 cement concrete. In spans over 150 feet in length the coping may require reinforcement. The top projecting edge should be chamfered.

Allowance for superstructure expansion.

9. Care must be taken that the superstructure can expand and contract without injuring the spans, abutments, piers, parapets, kerbs, or handrails.

In all slab, beam, and truss spans, provision must be allowed for an expansion of 1 inch per 100 feet of span. Expansion joints in bridge decking or floors are shown on Plate XXII; they are shown for the different types of bridge spans on the plates concerned.

Care must always be taken that the bearing and seating surfaces are smooth, to prevent binding or interlocking. Tar paper or felt expansion sheets are sometimes used in addition between the span bearings and the abutment or pier seatings to ensure that binding does not occur.

NORMAL TYPES OF FLAT BRIDGE SPANS UP TO 50′.

Rolled steel joist span with reinforced concrete floor.

10. Constructional details for rolled steel joist spans with reinforced concrete flooring are shown on Plate XIX.

This type is suitable for 12 ft. to 50 ft. spans.

The arrangement of joists shown allows of simple reinforcement details for the floor slabs. The use of fewer joists at larger intervals affords but little saving in their weight and consequent

cost, and necessitates the slabbing being reinforced for continuity.

Details for abutments, piers, and wing walls, are given in Plates VI, XVII and XVIII.

An example of this type is given in Chapter VII (Example 6).

11. Constructional details for steel troughing spans are shown on Plate XXI. Steel troughing span.

This type is suitable for 2 ft. to 22 ft. spans.

The economic span for a series of spans—$1\frac{3}{4}$ ft. (see Chapter V—Economic Spans).

The properties of steel troughing are given in Table XX. Steel troughing is a simple but expensive bridging material. Reinforced concrete decking is much cheaper, and should be used in preference when possible.

The wieght of troughing varies chiefly according to the thickness of the plate, and very little with respect to its depth. But as the strength varies directly as the depth, the most economical sections are those of the greatest depth for a given plate thickness, *e.g.* $6\frac{1}{2}'' \times \frac{5}{16}''$, and $15'' \times \frac{3}{8}''$, $\frac{7}{16}''$ and $\frac{1}{2}''$. For general use $6\frac{1}{2}'' \times \frac{5}{16}''$, and $10''$ or $12'' \times \frac{3}{8}''$ sections are the most suitable, and will take spans of 2 to 25 feet. The minimum suitable sections are, of course, shown in the design.

Troughing spans longer than 22 ft. may be built of longitudinal troughing filled with 1:2:4 cement concrete reinforced by bars in the bottom of the troughing. The strength would be the strength of the troughing *plus* that of the reinforced concrete slab.

An example of this type is given in Chapter VI (Example 3).

12. Constructional details for rolled steel joists spans with trough flooring are shown on Plate XXI. Rolled steel joist span with troughing floor.

This type is suitable for 12 ft. to 26 ft. spans, It is intended for use when only small troughing sections are available but is uneconomical, and either of the two preceding types should be used, in preference.

13. Each R. S. Joist should be ordered to the full length in one piece for each span. Joists are usually stocked up to 40 ft. lentghs. Rolled Steel Joist Splices.

Splices should be avoided if possible but one per span may be allowed in each R. S. Joist if thoroughly well made and riveted.

Where splices have to be used, each span length of joist should consist of the maximum length procurable, *plus* a short length to complete the span.

In the case of a series of spans, joists of the maximum length can be spliced together, and run continuously over the piers, up to 100 ft. total length.

In all cases splices in adjoining joists must be staggered at a distance apart=not less than one quarter of the span.

The splice will decrease the section modulus of each joist by about 10 per cent. but this need not be allowed for, as a splice should never occur at the centre of the span.

Splices must be made to give one inch of camber, and the ends of the joists must be cut and butted truly accordingly.

The arrangement of a splice is shown on Plate XIX.

An example of a R. S. Joist splice is given in Chapter VII (Example 7).

Reinforced concrete slab span.

14. Constructional details for reinforced concrete slab spans are shown on Plate XXII.

This type is suitable for 2 ft. to 15 ft. spans.

The economic span for a series of spans=$1\frac{1}{4}$ H (See Chapter V).

For small culverts, spans can be made of premoulded slabs, of a suitable size and weight for the type of transport available. The width of each slab should be a multiple of the main bar spacing. The slabs should be moulded with a key joint as shown, and grouped in position. In moulding, the tops of the slabs must be distinctly labelled.

Details for abutments, piers, and wing walls are given on Plates VI, XVII and XVIII. An example of this type is given in Chapter VII (Example 2).

Reinforced concrete beam span.

15. Constructional details for reinforced concrete beam spans are shown on Plate XXIII.

This type is suitable for 15 ft. to 50 ft. spans.

The economic span for a series of spans=$1\frac{3}{4}$ H (See Chapter V).

Details for reinforced concrete beams are given in Table XXIV.

The 3 beam arrangement shown is the most economical in material and form work, and gives simplicity in the floor slab and beam reinforcement.

All steel reinforcement must be ordered to suitable lengths and not by weight or lineal feet. Splices in beam bars may be allowed if staggered, the two ends being hooked and inter-locked, and wired to a tight bearing on each other. For a series of spans the beams may be made continuous, the continuous reinforcement over the piers being arranged as for slabs (See Chapter V, and Plate XI and Table XII).

End spans, being semi-continuous, may be reduced to 9/10 the depth *or* 9/10 the steel reinforcement given in Table XXIV, and intermediate spans may be reduced to 8/10 the depth *or* to 8/10 the steel reinforcement given in the table.

Details for abutments, piers, and wing walls are given on Plates VI, XVII and XVIII.

Examples of this type are given in Chapter VII (Examples 4 and 5).

Trestle pile bridges and overflow bridges.

Trestle Pile Bridges.

16. Trestle pile bridges are suitable where piles can be driven, and a headway not greater than 20 feet above the river bed is required, and are cheaper than other types if the necessary supervision, skilled labour, and plant are available.

They have the special advantage, apart from cheapness, of not obstructing the waterway. Subject to the conditions mentioned above they are suitable for low crossings over torrential nala streams in the hills, with beds of gravel and small boulders, where the current varies from a trickling stream, or nothing at all, to a torrent from 4 to 6 feet high, which arrives at short notice and ceases after a few hours. In such cases adequate protection against the boulders which the spates bring down must be provided above the trestles. A suitable method is to securely affix boulder crates upstream of the trestles, and also sometimes to provide guard piles, which may themselves be similarly strengthened.

The superstructure may be of reinforced concrete slabs or steel troughing on rolled steel joists, or reinforced concrete beams and decking according to materials available and individual practical considerations, having regard also to the suitability of the type selected for the length of span, which is limited by the strength of the piles and trestle considered in conjunction with the load which the bridge has to carry.

The piles may be of steel or reinforced concrete.

A homogeneous reinforced concrete structure is the ideal.

Timber pile bridges have been known to stand for many years, but they are not sufficiently permanent for ordinary use as permanent bridges.

Reinforced concrete Trestle Bridges.

17. Constructional details for reinforced concrete trestle spans are given in Plate XXV. Proportions of reinforced concrete piles are given in the table on Plate XVIII.

This type is suitable for 12 ft. to 20 ft. spans (15′ maximum when slabs only are used for the superstructure).

The economic span for a series of spans in a pile bridge = $\frac{3}{4}$ H for concrete slab decking, or H for flooring and beams (see Chapter V). For masonry (or concrete) piers, which may have to be used where piles cannot be driven, and in which term abutments are included as these may be of masonry or concrete, the economic span = $1\frac{1}{2}$ H for slab decking, or $1\frac{1}{4}$ H for flooring and beams.

In a series of spans the reinforced concrete floor slab, if used, may be made continuous, and reinforced over the piers as for continuous slabs (See Chapter V, and Plate XI and Table XII).

For end spans the average of columns 9 and 11 or 10 and 12 on Table XII should be taken, for intermediate spans columns 10 or 12.

Piles.

18. A pile "bent" is a transverse line of columns or piles supporting a bridge; a "tower" is two or more bents braced together.

In gravel and boulder beds reinforced concrete piles should be moulded with steel shoes. Reinforced concrete piles should be cured for not less than 30 days, and should be supported at the quarter points when lifted or moved; they should be designed for these latter conditions.

Piles must be driven below bed level to a depth not less than twice the maximum depth at Observed High Flood, ordinarily subject to a minimum of 10 feet. Their projection above the bed should not be greater than the buried depth.

If an outcrop of rock or very hard material, into which the piles will not sink, is met at a lesser depth than 10 feet, it will usually be necessary to use thin masonry (or concrete) piers.

The methods of driving piles and calculating the safe loads on piles are given in paras 62-66.

For reinforced concrete trestle pile bridges, the safe loads on the piles as so calculated in each case must not exceed the loads indicated on Plate XXV. If necessary the number of piles in each bent must be increased until calculations based upon the pile driving results show that they will carry the bridge load.

Piers and Abutments.

19. When masonry or concrete piers and abutments are used, their shape, etc., should conform to the ordinary standards given in Plates VI, XVII and XVIII, subject to the following:—

Top thickness of piers, in lime masonry $= 1' + \frac{\text{span}}{20}$

Ditto, in masonry in cement or cement concrete $= \frac{3}{4}' + \frac{\text{span}}{25}$

Ditto, in reinforced concrete, which should have 25 per cent. steel reinforcement in each face $= \frac{1}{2}' + \frac{\text{span}}{40}$

The piers should be stepped out below bed level so that the thickness of a pier is never less than—

$h/4\frac{1}{2}$ for lime masonry piers.
$h/6$ for masonry in cement or cement concrete piers.
$h/9$ for reinforced concrete piers

where h is the depth below the superstructure slab bearing or the top of the superstructure beams.

The depth of the piers below average bed level should generally be 1·6×maximum depth or 2·5×average depth at observed high flood, whichever is greater (see calculations for scour and foundations of bridges. Chapter V). The width of the piers at foundation level should generally be about 1/3 of their total height.

The shore spans may be supported on ordinary masonry or concrete abutments, or upon pile bents filled in with masonry or concrete walling.

20. Overflow Bridges are low bridges, with the superstructure at a low level, so that the normal flow or a small flood passes through the bridge, and a larger flood over the roadway. They are provided in conjunction with causeways, across the deepest channels in the river bed, causeways being given to link them up with the banks. Overflow Bridges.

Overflow bridges are suitable for use across straggling nalas, in lieu of causeways, where traffic would be unduly interrupted across an ordinary causeway crossing, and complete bridging above flood level would be unduly expensive. They are not suitable :—

(i) Across streams subject to floods of long duration or high velocity, or exceeding about 6 feet in depth.
(ii) Where the floods bring down large boulders or logs, etc.
(iii) In heavily silting river beds.

In cases where piles can be driven, the comparative cost of a trestle pile bridge above flood level from bank to bank, and of an overflow bridge cum causeways, should be worked out before it is decided to provide the latter.

Overflow bridges should be constructed similarly to trestle pile bridges, with the following special differences :—

The clearance between the lowest part of the nala bed and the superstructure should be that necessary to take an ordinary flood subject to a minimum of 3 feet. The ends of the bridge may be sloped up at a slope of 1 in 14 to a rise of 2 or 3 feet, giving a larger clearance (5′ to 6′) in the centre. In this case the changes of slope should be eased off as laid down for causeways (See Chapter VIII).

The roadway should have a cross slope of about 1 in 30 downstream.

The flooring should consist of reinforced concrete slabbing, without any covering, except a top dressing of asphaltic concrete (See para. 47). Beams should not be used, as they thicken the superstructure and increase the obstruction.

Handrails and kerbs, where necessary, should be of minimum dimensions, to afford the least possible obstruction to the flow, and must be well fixed to the bridge.

Where masonry (or concrete) piers are used (on beds, or portions thereof, where piles cannot be driven) they must not be built of lime masonry.

The maximum span is 15 feet (*vide* para. 13).

The economic span for a series of spans$=\frac{3}{4}$ H using pile bents, or $1\frac{1}{2}$H using masonry piers.

End spans of overflow bridges across midstream channels, adjoining causeways, should be supported on end cross-walls terminating the causeways, of depth equal to pier foundation depths, and stepped into the causeway drop walls or *vice versâ*.

Constructional details for overflow bridge spans are given in Plate XXV and Table XXVI.

PLATE GIRDER AND TRUSS BRIDGES.

Plate Girder and Truss Spans. 21. Plate girders are suitable for 40 ft. to 60 ft. spans.

Trusses are suitable for 60 ft. to 400 ft. spans.

These spans may be of the deck type with the girders or trusses under the roadway, or of the trough type with the roadway between the girders or trusses. The deck type is generally the cheaper.

For an 18 ft. roadway, deck type, the girders or trusses should be at 13 ft. centres, the roadway being cantilevered out on the overhanging floor beams. The distance between the trusses centre to centre, should be about $\frac{2}{3}$ the overall width of the deck.

In the through type the distance between the trusses is of course governed by the clear width of the roadway *plus* kerbs. The principles governing the design of truss bearings, and the consequent pier and abutment widths and seatings, are given in para. 8, and the necessary allowances for expansion are referred to in paras. 8 and 9.

Constructional details for abutments, wing walls, and piers are given in Plates VI, XVII and XVIII, and the relevant preceding paragraphs of this Chapter.

This type of bridge is that commonly used for large major bridges on Military Frontier roads in India, largely on account of the fact that old railway girders can be obtained at comparatively small cost and remodelled for use as road spans.

The flooring may consist of :-

(i) reinforced concrete slabs on reinforced concrete beams or rolled steel joists or rails, or

(ii) steel troughing on rolled steel joists, or

(iii) steel troughing alone, carried on the plate girders or trusses.

In the case of plate girder spans, when flooring system (i) is employed, the floor beams should be spaced 5′ to 10′ apart, with a longitudinal slab of suitable strength spanning between the

beams, no stringers being required. When steel troughing is used under system (ii), the floor joists should be spaced at the maximum span for the troughing.

In the case of truss spans, under systems (i) and (ii), the floor beams or joists should be placed at the panel points, the proportions of the decking being designed accordingly for the load, and stringers may be necessary for system (i). It is not economical or good engineering practice to apply the load elsewhere than at the panel points on the truss; if in any case this has to be done (as under system (iii)), the chord must be designed for the bending moment and direct stress, or in the case of old girders it must be verified that the stress can be borne by the chord (old railway girders such as are used in such cases will usually be of ample strength).

Except where the old girders are extemporized for the purpose, for efficient design not more than two plate girders or two trusses should be used per span.

Reinforced concrete arch bridges.

22. Reinforced concrete arched bridges constitute an important development of reinforced concrete construction. **Reinforced concrete Arch Bridges.**

The theory and principles governing their design and construction are here explained, and practical examples of two types of such bridges, *viz.*, with 100 ft. open spandrel arch spans and with 100 ft. filled spandrel arch spans, are worked out in Chapter VII, constructional details and type designs being shown in Plates XXVIII and XXIX

The terms applied to the various parts of an arch, together with constructional proportions and stress formulae according to the method here described, are shown in Plate XXVII.

23. In all arch bridges, the arches in which depend for their proper action upon horizontal and vertical resistance without settlement, rigidity of abutments, piers and foundations, is particularly essential, and care must be taken that the foundations are built on hard unyielding material, or supported on well foundations or piles at 3 or 4 ft. centres, also the unit stresses on the foundations must be safe. **Piers and Abutments.**

The abutments and piers in reinforced concrete arch bridges form one structure with the arches, and all arch reinforcement must be taken well down into them, to ensure continuity at the arch springings.

The design must be such that the thrust from the arch or arches.

(i) as for one span loaded and the other not loaded in the case of an unreinforced pier (*i.e.* the unbalanced live load thrust).

(ii) as for the shore span loaded in the case of an abutment, does not pass outside of the middle third.

Spandrels.

24. Spandrels may be open or filled.

Open spandrels consist of columns built on the arch ring or arch ribs, supporting the floor carrying the roadway. Spandrel columns should be spaced at $\frac{1}{8}$ to 1/16 of the span or 5 to 10 ft. apart, in small and large spans respectively.

Filled spandrels are filled solid (*e.g.*, with earth), side walling being provided between the arch and the flooring.

Principles of economic design.

25. For economy, the springing line should be placed as near the foundations as conditions will permit. This reduces the cost of piers and abutments.

The rise of the arch should be as large as possible, in order to reduce the material in the arch ring or ribs, piers, and abutments.

The filled type of spandrel is usually more economical than the open type, for arch rises under 15 ft., except when the foundations are not good, in which case the use of an open spandrel may reduce the cost of the abutments and piers, on account of the smaller dead load involved.

In open spandrels not more than two arch ribs should be used. A larger number than this complicates the floor construction, and gives no appreciable saving in its cost; it also increases the cost of the spandrels, the arch ribs, and the abutments. Two ribs only have been used for roadways 40 to 50 ft. wide.

The arch axis curve should ordinarily coincide with the equilibrium polygon for dead load; in rises greater than $\frac{1}{5}$ of the span it should coincide with the equilibrium polygon for dead load *plus* half live load. A curve of unfirm radius will seldom comply with this condition. A 3 centred curve can often be used.

Constructional details.

26. In arch ribs, the width of the section at the crown should be about twice the depth. As the width is kept constant, the section will approximate to a square at the springings. This arch rib shape has the advantage of giving low temperature and arch shortening stresses, but it involves a slight increase in sectional area over that required in narrow and deep sections. Narrow and deep sectional arch ribs have an unpleasing appearance.

The steel reinforcement percentage on the surface area of the arch rings or ribs should not be less than 0·5 per cent. in each face, at the crown and at the springings. As the arch rings or ribs are in compression, the top and bottom reinforcement layers must be effectively connected by steel bar binders placed 12″ to 18″ apart as in columns, to prevent the main reinforcement bars from breaking outwards. Arch ribs should be braced at intervals not exceeding 8 times the width of the rib. If the floor system is immediately above the arch, no braces are required at the crown, as the flooring braces the arch ribs effectively.

To avoid cracks in the spandrels due to the movement of the arch ring under live load and temperature stresses, or due to the deflection under dead load if the spandrels have been built before the centering has been removed, expansion joints must be provided in the spandrel walls, and in the floor of open spandrel bridges. These joints are usually placed symmetrically about 30 ft. apart, allowing for about 1″ of movement in either direction.

The centering must be rigid, and not liable to appreciable deflection during the construction of the arch. Small arches are generally concreted simultaneously from both springings to the crown, at a uniform rate. Larger arches are concreted first at the springings, then at the crown, the haunches being filled in last; this method reduces the distortion of the centering under construction loads, and the stresses due to shrinkage of the concrete.

The depth from the road surface to the top of the arches should not be less than 2′.

27. As concentrated live loads are effectively distributed by the earthfill, or by the rigidity of the floor slabs, the live load, in designing the arch ring, may be taken as uniformly distributed on the roadway. The uniform live loads given in Chapter V, para. 31, should be used for the arch ring. **Live Load.**

The floor system and spandrel columns should be designed for concentrated loads.

28. Reinforced concrete arches are usually designed according to the elastic theory (see para. 36). Such methods determine directly the position of the resistance line which gives the least average stress on the arch ring. The correctness of the elastic, theory has been proved by practical tests. **Standard methods of arch design.**

The graphical method is employed to determine, by trial and error, the position of the resistance line which gives the least average departure from the arch axis.

The variation in the stresses found by the two methods is about 10 per cent.

29. The standard methods of analysis according to the elastic theory are very laborious. The method here adopted, devised by Major Gibb, which is based on the analysis of elastic theory results by standard authorities (See Waddell's "Bridge Engineering" and Hool's "Concrete Engineer's Handbook," Cochrane's analysis), will be found suitable in practice for both large and small spans, provided that:— **Approximate Practical Method.**

(i) (*Vide* para. 25). The arch axis coincides with the resistance line or equilibrium polygon, for dead load in the case of small rises and for dead load *plus* half live load in the case of rises greater than $\frac{1}{8}$ of the span.

(ii) The depth or thickness of the arch ring or rib increases from the crown to the springings in the proportions laid down in the method (see Plate XXVII).

(iii) (*Vide* para. 25). The steel reinforcement percentage on the calculated area is not less than 0·5 per cent. in each face, at the crown and springings.

(iv) (*Vide* para. 25). The depth between the road surface and the top of the arch is not less than 2′.

Notation. 30. The notation adopted in the formulæ is as follows. (See Plate XXVII.)

All units feet and pounds.

w_c=dead load per lineal foot at crown.

w_s=dead load per lineal foot at springings, clear of abutment and details.

$$\mu = \frac{w_s}{w_c}$$

w=live load per lineal foot.

(*N. B*—w_c, w_s and w are usually taken on a 1 ft width for arch rings, and on the width carried by one rib for ribs).

L=span of arch axis.

r=rise of arch axis.

x & y=abscissae and ordinates to arch axis.

Angle β=inclination of springing line to horizontal.

h_c=crown thickness or depth of arch ring or rib (in feet or inches).

h_s=springing thickness or depth of arch ring or rib (in feet or inches).

M_c=moment at crown.

M_s=moment at springing.

T_s=thrust at springing.

V_s=shear at springing.

b=width of arch ring or rib considered, usually 1 ft. for rings, or width of rib for ribs.

H=horizontal thrust consequent on arch action: for any given loading H is uniform throughout the arch.

$\pm t°$=rise or fall in temperature in degrees Farenheit.

For spandrel filled arches t is taken=$\pm 20°$; for open spandrels $\pm 30°$.

H_s=horizontal thrust due to dead load+half live load +temperature.

e=excentricity of the thrust from the arch axis=$\frac{\text{moment.}}{\text{thrust.}}$

31. The curve of the arch is found by the following (Waddell's) formula (which is suitable for all arches):— **Arch proportions.**

$$y=\frac{4r}{L^2(u+5)}\left[6x^2+(u\text{-}1)\frac{4x^4}{L^2}\right] \quad . \quad . \quad . \quad . \quad (1)$$

$$\text{where } u=\frac{w_s}{w_c} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

The inclination of the springing line to the horizontal is found by the equation

$$\tan\beta=\frac{4r.}{L^2(u+5)}[6\,L+2L\,(u-1)] \quad . \quad . \quad . \quad (3)$$

The curve may also be determined graphically. A 3 centred curve will usually fit the plotted points. The equations for the radii of this curve are given on Plate XXVII, *viz.*:—

$$R=\frac{(ad)^2+(bd)^2}{2bd} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

$$R_1=\frac{(ef)+(ed)^2}{2\,(ed\cos\theta-ef\sin\theta)} \quad . \quad . \quad . \quad . \quad . \quad . \quad (5)$$

$$\text{where }\sin\theta=\frac{ad}{R} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (6)$$

The depth of the arch ring or rib increases from the crown to the springing as follows (see Plate XXVII).

Depth at crown $=h_c$ ft. or ins.

Depth at springing $=h_s$ ft. or ins.

Depth at $\frac{1}{4}$ way from crown $=\cdot 89h_c+\cdot 11h_s$ . . . (7a)

Depth at $\frac{1}{2}$ way from crown $=\cdot 78h_c+\cdot 22h_s$ . . . (7b)

Depth at $\frac{3}{4}$ way from crown $=\cdot 60h_c+\cdot 40h_s$ . . . (7c)

Approximate values for h_c and h_s are as follows:-

$$h_c=\frac{(\text{span in ft.})^2}{9600}+\cdot 66\text{ ft.} \quad . \quad . \quad . \quad . \quad . \quad . \quad (8)$$

$$h_s=h_c\sec.\ \beta. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (9)$$

A trial design for the arch should be made accordingly, and verified or altered as necessary as a result of the application of the formulae for stresses and moments, which are as follows (see Plate XXVII).

All units in these formulae are feet and lbs.

32. The dead and live load thrusts are due to arch action, and the moments are induced by similar conditions to those in continuous beams. **Load Stresses.**

(i) For dead load (applicable to crown and springing).

$$H=\frac{w_cL^2}{48r.}(u+5) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (10)$$

$$M_c=\pm\frac{H\,h_c}{40} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (11)$$

$$T_s=H\sec.\ \beta \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (12)$$

$$M_s = \pm \frac{T_s\, h_s}{40} \qquad \text{(13)}$$

$$V_s = \frac{W_c\, L}{6}(u+2) \qquad \text{(14)}$$

(ii) For live load for Max. $\pm M_c$ (applicable to crown)

$$H = \frac{w\, L^2}{16\, r.} \qquad \text{(15)}$$

$$M_c = \pm \frac{w\, L^2}{110} \qquad \text{(16)}$$

(iii) For live load for Max. $+M_s$ (applicable to springing).

$$H = \frac{w\, L^2}{10\, r.} \qquad \text{(17)}$$

$$T_s = H \sec \beta \qquad \text{(18)}$$

$$M_s = +\frac{w\, L^2}{40} \text{ for open spandrels} \qquad \text{(19a)}$$

$$= +\frac{w\, L^2}{30} \text{ for filled spandrels} \qquad \text{(19b)}$$

$$V_s = \frac{wL}{5} \qquad \text{(20)}$$

(iv) For live load for Max. $-M_s$ (applicable to springing).

$$H = \frac{w\, L^2}{30\, r.} \qquad \text{(21)}$$

$$T_s = H \sec \beta \qquad \text{(22)}$$

$$M_s = -\frac{w\, L^2}{50} \qquad \text{(23)}$$

$$V_s = \frac{w\, L}{3} \qquad \text{(24)}$$

(v) For live load over whole span.

$$H = \frac{w\, L^2}{8\, r.} \qquad \text{(25)}$$

$$V_s = \frac{w\, L^2}{2} \qquad \text{(26)}$$

Temperature Stresses.

33. For a rise in temperature, the length of the arch axis increases and the crown rises, producing a horizontal thrust on the abutments, a negative moment at the crown, and a positive moment at the springing. For a fall in temperature the conditions are reversed.

For temperature $\pm$ t° (applicable to crown and springing).

$$\text{Alteration in } H = \pm \frac{4500\, b.\, h^3\, t^\circ}{r^2} \qquad \text{(27)}$$

where $h = h_c$ or h_s

$$\text{Alteration in } M_c = \pm \frac{H\, r}{4} \qquad \text{(28)}$$

$$\text{Alteration in } T_s = \pm \frac{H}{\text{Sec } \beta} \quad \dots \dots \quad (29)$$

$$\text{Alteration in } M_s = \pm \frac{3 H r}{4} \quad \dots \dots \quad (30)$$

34. The effect of compressive stress in the arching due to dead and live loads and temperature is to reduce its length, producing results similar to those due to a fall in temperature. Arch Shortening Stresses.

For arch shortening (applicable to crown and springing).

$$\text{Alteration in } H = -\left(1 + \frac{\cdot 3\, h_s^3}{h_c^3}\right) \frac{H_a\, h_c^2}{r^2 \text{ Sec } \beta} \quad \dots \quad (31)$$

where H_a=H obtained by equation (10)+$\frac{1}{2}$H obtained by equation (25) +difference in H obtained by equation (27).

$$\text{Alteration in } M_c = + \frac{H r}{4} \quad \dots \dots \quad (32)$$

$$\text{Alteration in } T_s = - \frac{H}{\text{Sec } \beta} \quad \dots \dots \quad (33)$$

$$\text{Alteration in } M_s = - \frac{3 H r}{4} \quad \dots \dots \quad (34)$$

35. An implied excentricity of $\frac{hc}{40}$ and $\frac{hs}{40}$ is assumed for the dead load moments M_c and M_s (Cochrane). Remarks on the method.

This method will never give results smaller than those derived by the standard elastic theory methods.

The values of H T_s and V_s are correct for dead load.

For live load, temperature, and arch shortening H may be 5 per cent. to 10 per cent. in excess, and M_c T_s and M_s 10 to 30 per cent. in excess of the accurate values obtained by the elastic theory depending upon the proportions of the arch. Such errors, however, are so minimized by the dead load forces, which are much greater than the others, that the resulting arch sections will not vary from those obtained by the arduous methods of elastic theory analysis by more than 10 per cent. in the worst cases and about 5 per cent. in average cases.

In this method, the same convention is used as in beams, *i.e.*, +M gives the tension in the bottom surface, and —M gives the tension in the top surface. Shear stresses need not be investigated, as they are never high, and the concrete and the bar binders between the top and bottom layers of steel reinforcement (see para. 26) furnish ample strength to withstand them.

This method can be used for the design of masonry or unreinforced concrete arches (see para. 3), in which case the unit stresses must be safe for the material and the thickness of the arch ring must be not less than 4 e to 6 e, say 5 e.

36. If for any reason the arch axis cannot be made to coincide with the resistance line or equilibrium polygon for the load, as Method for Special Cases.

defined in para. 25, the arch must be analysed by one of the standard elastic theory methods, *vide* "Principles of Reinforced Concrete Construction," Volume III, by Hool, or "Concrete Plain and Reinforced" by Taylor and Thompson.

It is usually sufficient to find the maximum stresses at the crown and springing of the arch, and at a point $\frac{1}{5}$ of the span from the crown.

An approximate method would be to use the same values of H M_c T_s and M_s for live load, Temperature, and Arch Shortening as shown on Plate XXVII, and increase the Dead Load Moments M_c and M_s by H or T_s multiplied by the excentricity of the arch axis from the resistance line for dead load which has the least average departure from the arch axis.

Examples. 37. A practical exposition of the method of design outlined above, including further details and practical notes on the construction and design of this type of bridge, is given in the examples of complete reinforced concrete arched 100′ span bridges of open spandrel and filled spandrel type worked out in Chapter VII (examples 9 and 10).

REINFORCED CONCRETE BOWSTRING BRIDGES.

Reinforced Concrete Bowstring Bridges. 38. Reinforced concrete bowstring girder spans constitute a useful type of bridging, in large bridges requiring small headway; they are thus particularly suitable for crossing torrential mountain streams.

An illustration of a single 315′ clear span road bridge of this type, as constructed in France, is given on Plate XXX.

SUSPENSION BRIDGES.

Suspension Bridges. 39. The modern type of stiffened suspension bridge is economically and practically suitable for highway road bridge spans, to carry M. T., of 300 ft. and upwards. Experience in America and France shows that the cost of this type in bridges of moderate span is not uneconomical as compared with other types, and in fact in many cases it is cheaper.

This type of bridge should therefore be studied, and its use considered, when designing large bridges for military roads in India.

The advantage of avoiding the construction of piers in the river bed is apparent in all cases; and the use of suspension bridges across the torrential nala crossings on Frontier hill roads, where practicable, is particularly attractive.

Modern highway suspension bridges (disregarding field suspension bridges, which are not dealt with here), are of the following types :—

(i) The wire cable bridge with separate steel stiffening truss, as used in America.

(ii) The wire cable bridge with inclined stays, as used in France and French colonies.

(iii) The chain or rivetted-chord bridges, as used in Germany and Austria.

The use of eyebar chains is preferable to steel cables, as they have the advantages of greater rigidity, and greater resistance against corrosion; they permit of efficient attachment of the stiffening web members, combined with which they form a homogeneous rigid main carrying system.

The bridge flooring should be as rigid as possible, *e.g.*, of reinforced concrete or steel, to assist in promoting the necessary stiffness.

For the design of Suspension Bridges, see "Notes on Applied Mechanics (structural)" published by the S. M. E. Chatham, Waddell's Bridge Engineering, and standard works on Suspension Bridges.

LIGHT BRIDGES ON PACK TRANSPORT ROADS.

40. On pack transport hill roads, as in Chitral, light timber bridges, using local materials supplemented by the necessary iron work, sleepers, and wire cables, are often provided, in the form of suspension bridges, cantilever bridges, tension bridges, and crib pier bridges **Light Bridges.**

The design of these bridges is described in the Manual of Military Engineering.

The loads which they should be calculated to take are given in Chapter V, para. 36.

The standard roadway dimensions are given in the General Specifications (Chapter III and Appendix III).

BOAT BRIDGES.

41. Boat bridges and ferries exist in many places on Indian roads, but they are not now provided as a new measure on high ways for M. T. traffic, except as field bridges on active service or as a temporary expedient. They are expensive in upkeep, and are liable to be washed away by heavy floods. **Boat Bridges.**

Where practicable, funds being available, existing boat bridges are replaced by pucca bridges, or strengthened as far as possible to take heavy M. T., where necessary and where their replacement is impracticable for financial or other reasons. In some cases (*e.g.*, on the river Indus crossing between Darya Khan and Dera Ismail Khan), where the provision of a pucca bridge is impracticable, boat bridges are maintained in the cold weather, and are dismantled and replaced by ferries in the hot weather flood season.

The methods of construction of boat bridges are given in Manual of Military Engineering. On Indian highways country boats or

special pattern boats (in the Punjab and North-West Frontier Province the Jhelum pattern is standardized) are used for pontoons, and trussed beams are used to carry the superstructure, other details of the equipment also being of local pattern.

BRIDGE DEFENCES.

Bridge Defences.

42. When it is necessary to provide bridge defences, as in the case of important bridges and boat bridges, particularly in Frontier districts, these usually take the form of diagonally opposite double story blockhouses located alongside the bridgeheads, suitable as a rule for garrisons of 6 to 12 men. The defences should be designed so as to afford free traversing fire along the bridge, combined with facilities for ordinary defensive fire in all directions over the roof parapet and through horizontal loopholes therein, and close-up protection by means of machicoulis and low vertical loopholes through the base of the roof parapet. Entrance is usually gained by a steel door in the upper story, with a draw-up ladder, and a barbed wire perimeter entanglement, with "mazed" entrance closed by a knife rest or gate, is provided. In certain cases, barriers or unclimbable gates may be necessary to close the bridge when required. In such cases, these should be worked in to the block-house defence schemes, and care should be taken that barriers or gates are properly commanded by fire from the blockhouses.

MISCELLANEOUS BRIDGE DETAILS AND SPECIFICATIONS.

Handrails and Posts.

43. Handrails and posts should be designed for a horizontal load of 75 lbs per lineal foot. Their height is given in the General Specification (Chapter II), *viz.*, 1½′ on culverts and 2½′ on bridges, increased to 3′ on long high bridges.

Steel handrails and posts should be designed as follows :—

Handrails.

Deflection $\frac{L}{360}$, where

L = distance between posts in ft.

$$I = \frac{W L^2}{20} = \frac{75 \times L^3}{20 \times 2240} = \frac{L^3}{600}$$

For distance between posts of 5½′ 6¼′ 7⅛′ 7¾′ 8⅝′ 10¾′

G. I. piping of the following diameters should be used 1¼″ 1⅜″ 1¾″ 2″ 2¼″ 2½″.

As a rule, 2 inch G. I. piping should be used for the top rail, with the posts at a maximum interval of 8 ft.

Posts.

Bending Moment $M = (75 \times L \times H)$ ft. lbs.

Where H = height from top rail to junction of post with bridge.

$$\frac{I}{Y} = Z = \frac{M}{f} = \frac{75 \times L \times H \times 12}{7\frac{1}{2} \times 2240} = \frac{L \times H}{18{\cdot}7}$$

For posts 8′ apart and $H = 3'$,

$$Z = \frac{8 \times 3}{18{\cdot}7} = 1{\cdot}28$$

$4'' \times 3'' \times \frac{5}{16}''$ angle iron is suitable, with the 4″ flange across the handrail.

Reinforced concrete handrails and posts are particularly suitable with reinforced concrete superstructures. Details of these are given in the type designs for reinforced concrete bridges.

44. Parapet copings should normally be of 1:2½:5 cement concrete or large dressed stones set in 1:2 cement mortar. All copings should be weather sloped on the upper surface. Parapet Copings.

It is sometimes advisable to adopt such devices as to bond in the end coping stones with iron bars let into the masonry and into the coping stones, to give greater stability against bumps and to render wilful removal more difficult.

45. Wheel guards or kerbs should be constructed of first class stone in cement or cement concrete. Wheel Guards.

46. The ordinary method of finishing off the roadway over reinforced concrete decking is to coat the concrete with asphalte and cover it with about 1½″ of clay and 6″ metalling, which gives a cheap and easily renewable wearing surface. Road surface on concrete decking.

The decking may, however, be covered in lieu with asphaltic concrete alone; this is the only method in the case of overflow bridges.

47. The following is a suitable method of asphaltic concrete surfacing. Asphaltic concrete surfacing.

Mixture.

	lbs.
Fine shingle	150
Sand	150
Asphalte	150
Bitumen	50
Portland Cement	50

The asphalte and bitumen are heated in a boiler until melted to the consistency of cream. The shingle and cement, having been mixed dry, are then added gradually and stirred in.

The concrete surface is then cleaned, and the mixture is laid and lightly rammed with wooden rammers to a thickness of 1 to 1½ inches.

The mixture when dry forms a hard surface which wears well under traffic, is not too brittle, and does not get too soft during the hot weather. The surface should be kept covered with a thin layer of sand to prevent it from getting slippery.

Sufficient boilers should be employed to minimize joints, and enable a large area to be laid at one time.

Where there are expansion joints in the girders and in the concrete decking, plain asphalte expansion joints must be given in the asphaltic concrete surfacing.

This method of surfacing concrete bridge decking has the additional advantage of reducing the weight of the superstructure and the consequent load on the girders, etc.

Roadway drainage.

48. Adequate drainage must be given to the roadway on all bridges and culverts. Drainage openings, about 6″ square, with their upper sills 3″ above the surface of the metalling should be given on both sides at intervals of 8 to 10 ft., provided where necessary with a projecting lip to throw the drainage clear of the structure. The metalling should be cut away and sloped down immediately in front of each drainage opening.

Specification for masonry work.

49. The materials and workmanship in all road structures (including bridges and culverts) must conform to the specifications laid down for building work generally (see Volume I.—Buildings and General) as applicable, amplified by the specifications given in this Chapter and in Chapters IV (Retaining Walls) and VIII (Causeways).

All road structure work demands the best materials and workmanship in an especial degree.

Lime and Cement.

50. Lime mortar and lime concrete may be used in bridges, culverts, causeways, etc., where the lime is hydraulic, the foundations are dry, and the river carries silt and sand only. In any case, lime should not be used in piers and abutments when the spans exceed 50 feet.

Cement mortar and concrete should be used where the lime available is not a first class hydraulic lime, or the foundations are wet, or the masonry is subject to the hammering action of gravel, or bajri and boulders. In spans over 50 ft., cement concrete and mortar should be used in any case, as the piers and abutments in large spans carry considerable loads, to meet which lime mortar and concrete are ordinarily not sufficiently reliable.

Piers and abutments in cement mortar or concrete can often be made of smaller dimensions than if lime were used ; this partially counterbalances the extra cost involved, and in some cases will cause an actual saving.

Mixed specifications in cross sections, such as lime concrete filling inside a cement masonry skin, are unsound and should not be adopted ; the strength of the cross section must be uniform.

51. Where lime concrete is permissible in foundations and base courses, or cores, the composition should be 1 part of lime mortar to $2\frac{1}{2}$ parts of clean coarse aggregate, by volume; the mortar should be composed of 1 part of hydraulic lime to 1 of sand and 1 of surkhi. **Concrete in foundations and base courses.**

Cement concrete for this purpose should be composed of 1 part of Portland cement to 3 of sand to 6 of aggregate (1:3:6). In very wet foundations the outer ring of the base may be made of cement concrete laid in bags, headers and stretchers alternately, well rammed into position.

52. Masonry piers and abutments for spans not exceeding 50 ft. (in cases where the use of lime is permissible), should be built of solid lime masonry consisting of hammer dressed coursed rubble stone, or brick. In all except small culverts not liable to severe stress first class bricks only should be used. The masonry should be pointed with 1:2 cement mortar from the base to high flood level, the joists being cleanly raked out $1\frac{1}{2}''$ deep, and wetted, before pointing. **Masonry of piers and abutments.**

In all spans exceeding 50 ft., and in smaller spans where cement is to be used, masonry piers and abutments should be built (i) of solid hammer dressed squared and coursed stones, or first class brick in 1:2 cement mortar or (ii) of an outer ring up to nala bed level of first class stone or brick masonry in 1:2 cement mortar, of thickness at least $\frac{1}{4}$ the width of the piers, filled with a core of 1:3:6 plum cement cencrete; the construction above nala bed level to be as in (i).

The plums should consist of large clean sound boulders placed by hand not closer together than 1 foot in all directions.

53. Superstructure seatings must be of 1:2:4 cement concrete of the following thicknesses in feet:— **Superstructure seatings.**

For slabs :—$\frac{1}{4}' + \frac{\text{span}}{80}$ subject to a minimum of 6″.

For beam and truss bridges on cement masonry piers and abutments:—$\frac{1}{3}' + \frac{\text{span}}{60}$.

For beam and truss bridges on lime masonry piers and abutments:—$\frac{1}{2}' + \frac{\text{span}}{40}$.

Superstructure seats must extend across the full width of the pier or abutment (see also para. 8).

54. Spandrel walls in masonry arched bridges should be built of hammer dressed coursed rubble stone or brick on the best lime mortar locally procurable. **Masonry Spandrel Walls.**

Arch rings in masonry bridges.

55. Arch rings in masonry bridges should be composed as follows :—

(i) Cement concrete 1-2-4, with stone or moulded cement concrete voussoirs on the faces (suitable for 2 ft. to 50 ft. spans),

or (ii) Chisel dressed voussoir stone in 1-3 cement mortar (suitable for 30 ft. to 50 ft. spans),

or (iii) Hammer dressed voussoir stone in first class lime mortar (suitable for 2 ft. to 25 ft. spans).

or (iv) 1st class brick in cement or lime mortar using specially moulded arch bricks as necessary (suitable for 2 ft. to 20 ft. spans).

or (v) 1st class lime concrete, with stone or moulded cement concrete voussoirs on the faces (suitable for 2 ft. to 10 ft. spans).

The centering for arch rings in lime mortar or concrete should not be struck for at least 28 days after the arch ring and haunches have been finished, or until the lime has set sufficiently.

The backfill should be completed to springing level at least, and preferably totally completed, before the centering is struck. (See also para. 25).

Masonry Arch haunches.

56. Abutment haunches for stone or brick arch rings should be built of the same material, with the courses approximately radial to the centre from which the arch is struck. Pier haunches for stone or brick arch rings should be of first class lime concrete.

Arrangements for bridge construction.

57. It is a first principle of sound engineering that full and systematic arrangements (" bundobust ") for the work shall be made before construction starts; including engineering staff, labour, materials, accommodation, and tools and plant.

Particularly in the case of a bridge, to start work without such arrangements having been fully made beforehand must always increase the difficulties of construction and add to the total expense, without saving time in the end.

Essential training and protection works must be built in the first instance—they may take precedence of all other arrangements.

Depth of foundations.

58. In all cases foundations must be taken to safe depths below scour, as laid down in Chapter V. The type of foundation adopted must be such as to comply with this condition, in the river bed concerned.

OPEN BRIDGE FOUNDATIONS.

Open Foundations.

59. Open bridge foundations, as in torrential stream boulder beds, are best excavated by using a crib framing, to ensure that they can be taken to the full depth. The crib should be strongly made of 2" or 3" timber, well braced and put together, and smooth on the outside (an important detail)

It is sunk as the material is excavated from the inside, the excavation being kept unwatered by suitable pumps. Crib work should never consist of odd pieces of timber roughly put together; an inferior crib will cost more in delays through refusal to sink and from material entering under it than the cost of a properly made crib, and delays are matters of considerable importance when working in a nala bed, with the prospect of floods destroying the work before it is flood proof. The same set of crib work, or sets, is or are successively used for successive piers or groups of piers.

The excavation of open foundations without the use of any crib work should be avoided. It costs a great deal more in excavation, and is an undesirably uncertain method when the conditions of bridge foundation work are considered.

WELL FOUNDATIONS AND WELL-SINKING.

60. In silty river beds free from boulders, masonry wells with a steel, timber, or reinforced concrete curb or cutting edge may be used. The material is excavated from the inside of the well by a clam shell or grab bucket dredger. Well foundations.

The skin friction encountered in sinking the well varies with the character of the material of the well, the strata penetrated, and the depth. The average skin friction in lbs. per sq. ft. of surface (A) for any depth h ft. appears to vary roughly as xh, where x varies from 5 to 15 for sand and clay, and from 2 to 8 for mud and silt; the total resistance of the well to sinking Axh lbs. This rule is only given as a rough guide; the results obtained in sinking should be used finally.

The curb increases in thickness inwards to provide a seating for the masonry. In hard river bed material the curb must be made of steel, or of reinforced concrete with a steel cutting edge; in soft material reinforced concrete can be used without protection. Timber can be used for unimportant work, with a steel cutting edge when necessary as in the case of concrete. Well curbs.

The thickness of the masonry shell or steining should be about $2\frac{1}{2}$ ft. Where high skin friction is anticipated, thick steinings may be used, with a view to the additional weight facilitating the sinking of the well without the addtion of a super-imposed load for the purpose. Steining.

The shaft of a well should be from 8 to 10 ft. in diameter, to give room for operating the excavating gear.

From the curb, vertical rods securely fastened to it at about 3 ft. centres are brought up through the centre of the masonry steining. These rods are tied together at vertical intervals with flat iron, which is securely fastened to the rods by threading the end of each rod and bolting it securely in place with a sleeve nut or otherwise.

This ensures that the masonry will sink uniformly, and not be held up by increased skin friction at any point.

When the well has reached an impenetrable stratum, or alternatively has been sunk to a sufficient depth to carry a test load satisfactorily by means of the resistance set up by the skin friction of the material on the sides of the well (subject always to the requisite depth below scour having been attained), the bottom of the well is plugged with several feet of cement concrete. The remainder of the well is then filled up with concrete, and a cement concrete cap is placed on the top to distribute the load.

In excavating, the depth of excavation below the cutting edge of the well curb should never exceed 4½ ft. in a sandy bed, or a "blow" is certain to take place, tending to throw the well out of the vertical. When a well sticks in sinking owing to increasing hardness of, or friction with, the river bed material, a small charge of dynamite (2 ozs. or less), placed in the centre, should usually suffice to get the well on the move again.

Before the first layer of masonry is built, the space inside the curb should be filled up with sand, to minimize tilting.

Rapid method of well sinking.

61. A rapid method of constructing well foundations in a running stream with a bed of sand, shingle, and small boulders, is that adopted in the case of the bridges across the Kabul river at Michni near Peshawar, as follows.

The river bed was first piled. Each well curb, with brick steining super-imposed, was then lowered, 1 foot at a time, by means of a suspension and lowering device The curb was forced down by means of 14 heavy screw jacks of 20 tons capacity each, until the curb founded squarely on the river bed. The well was then sunk by dredging through the boulders and shingle of the river bed. A large quantity of boulders had to be broken up by blasting before the curb reached its final level. The curb was made up of heavy steel plating and angle sections, to resist the concussion caused in blasting out the boulders. The brick work steining was built up above water level concurrently with the sinking of the curb. When the well had been sunk to foundation depth (about 12′ in this case). the inside was plugged with concrete.

This method is described as being suitable to the conditions mentioned: it is not recommended where deeper foundations, in a shingle and boulder bed, are required– the foundations would have been taken deeper in this case had not the increasing difficulty of sinking rendered this impracticable.

PILES AND PILE DRIVING.

Piles.

62. Piles are ordinarily made of timber, reinforced concrete, or steel, according to availability of materials, the importance of the work, and the nature of the river bed.

Where timber poles are used for piles, they should be of diameter 6 to 8 inches at the tips, 14 to 18 inches at the butts, and taper uniformly. They must be cut from sound trees free from unsound or loose knot, etc. and should not vary from the straight by more than half the middle diameter. The bark should be peeled off and all branches trimmed off completely. The point should be accurately central and shaped conically. Except in soft ground, the point should be protected by a wrought or cast iron shell, or by a couple of V shaped laaps of bar iron spiked to the pole.

Concrete piles should be designed in accordance with the details shown on Table XXVI. They should be made of 1:2:4 cement concrete, cured for at least 30 days, and when lifted or moved should be supported at the quarter points in their length. They should be fitted with steel shoes, except when the ground is soft and free from boulders and shingle (see also paras. 17 and 18).

63. Piles may be driven :— **Pile Driving.**

(i) by a drop hammer,

(ii) by a steam hammer, single or double acting,

(iii) by a percussion pile hammer.

In hard ground a percussion hammer will sink a pile which would be crushed if driven by a drop hammer. The drop hammer should weigh not less than the weight of the pile.

The tops of the piles should always be protected by cushions of wood or rope, whilst they are being driven.

In driving, if the hammer bounces to any extent, the fall must be too great, the hammer must be too light, or the pile must have struck an impenetrable obstacle. Decreasing the fall will frequently increase the effectiveness of the blow and diminish the bouncing. If the point of the pile reaches an impenetrable stratum and hard driving is continued, the foot of the pile may be spread out and spoilt. When the penetration has diminished to ¼ to ½ inch per blow, and the hammer rebounds considerably, the safe limit of driving has been reached.

As a general rule, piles need not be driven more than 10 feet in hard ground, to 20 feet in soft ground, provided that the penetration under the last series of blows is satisfactory, and that scour depth conditions are met.

Piles may be badly damaged by over-driving ; hence once the penetration and depth are satisfactory, driving should be discontinued.

64. Where the pile passes through soft material and the lower end rests on an impenetrable stratum, the permissible load is limited to the strength of the pile considered as a column. **Safe loads on piles.**

The other case is that of a pile driven through fairly uniform material ; the load is then carried by the friction of the bed material

on the sides of the pile. and the permissible load is determined in relation thereto.

For a friction pile, the following formulae give the safe load. They include a factor of safety of 6, which it is usual to allow for vibration :—

For piles driven with a drop hammer.

$$P = \frac{Wh}{S+1} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

For piles driven with a single acting steam hammer,

$$P = \frac{2\,W\,h}{S+0{\cdot}1} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

For piles driven with a double acting steam hammer,

$$P = \frac{2\,h\,(W\,a.p\,)}{S+0{\cdot}1} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

where

P=safe load in lbs

W=weight of hammer in lbs

h=fall of hammer in ft

a=effective piston area in square inches.

p=effective steam pressure in lbs./sq. in.

S=penetration under the last blow, in inches: this may be taken as the average penetration per blow under the last 5 blows, where the penetration is small

Water Jet Method.

65 In sand, mud, or soft clay. piles may be founded by discharging 1, 2 or 3 water jets at the point of the pile, thus loosening the soil and enabling the pile to sink by its own weight, assisted if necessary by very light blows from a drop hammer.

In this method, the water is usually conveyed to the point of the pile through a pipe fastened to the pile; the jets must be in advance of the pile point.

The water jet method is almost useless in gravel, or in sand containing gravel and clay.

"Foundations" by Fowler should be consulted for full particulars.

Screw Piles

66. Screw piles generally consist of a rolled iron shaft, or a cast iron pipe, 3 to 10 inches in diameter, having at its foot one or two turns of a cast iron screw with blades 1½ to 5 feet in diameter. They are screwed into the ground by a hand-operated capstan, or other suitable mechanical or animal power method.

Screw piles will penetrate the majority of ordinary river beds; *e.g.* they can be driven into clay and loose pebbles and stones, and will, push aside small boulders. They are not efficient in dry sand and cannot be used in beds containing large boulders or petrified conglomerate, etc.

CHAPTER VII.

Bridges and Culverts—Examples.

Introductory—Calculations for Foundations, etc., for a Bridge across an average hill stream—6′ Reinforced Concrete Slab Culvert—6′ Steel Troughing Span—Reinforced Concrete Slab Decking Continuous over Longitudinal Reinforced Concrete Beams—Reinforced Concrete Beam Bridge—Reinforced Concrete Flooring on Rolled Steel Joists Bridge—Rolled Steel Joist Splice—Masonry Arch Bridge—Reinforced Concrete Open Spandrel Arch Bridge—Reinforced Concrete Filled Spandrel Arch Bridge.

INTRODUCTORY.

In this Chapter examples illustrating the application of the principles and methods described in Chapter VI, and utilizing the tabulated working formulæ, diagrams, and results, are given.

The standard load (12 ton road roller with 25% impact) is catered for in all cases.

For details of steel and reinforced concrete design, and the general principles thereof, reference should be made to the relevant chapters of Volume I (Buildings & General).

EXAMPLE 1.

Calculations for foundations, etc., for a bridge across an average hill Stream.

See Chapter V, paras. 9-27.

	Above Bridge site.	At Bridge site.	Below Bridge site.
Cross sectional area of waterway at O.H.F. $A =$ sq. ft.	2,900	3,000	3,200
Wetted perimeter corresponding to A, $P =$ ft.	410	400	433
Hydraulic mean depth below O.H.F. $r = \frac{A}{P} =$ ft.	7·1	7·5	7·4
Maximum depth to scour holes below O.H.F.L., D, = ft.	10·5	12·5	12·4
Slope of water surface, $S, = \frac{\text{fall}}{\text{distance}}$, =	·0074	·0070	·0067
Coefficient of rugosity, n =	—	·035	—
Hence Mean velocity of stream at O.H.F. $V = \frac{1·486}{n} \times r^{\frac{2}{3}} \times s^{\frac{1}{2}}$ ft. sec =	13·45	13·7	13·1
Discharge at O.H.F. $Q = A.V.$ cusecs. =	39,000	41,000	42,000

D

As the value of Q for (*b*) (Bridge site) is within 5°/$_o$ of the mean value of Q for sites (*a*) and (*b*), the calculations may proceed, utilizing the values for the bridge site.

R. L. of O. H. F. L. = 131·80.

R. L. of mean bed level = 124·30.

The catchment area discharging through this bridge, measured from the map, is 630 sq. miles approximately. The run-off, $Q = 2{,}100 \times M^{\frac{3}{4}}$.

= *53,000 cusecs approx.*

To allow for an abnormal flood, the discharge at maximum high flood must be taken approx :—this, subject to a minimum of $1\frac{1}{3}$ Q.

Take $Q_1 = 1\frac{1}{3}Q$ = *55,000 cusecs approx.*

The hydraulic mean depth at Max. High Flood

$r_1 = 1\frac{2}{3} \sqrt{1\frac{1}{3}} \times r$ approx.

$= 1{\cdot}19\ r$

$= 1{\cdot}19 \times 7{\cdot}5 = 8{\cdot}95$ *ft.*

The approx. rise in surface

$= r_1 - r = 1{\cdot}45$ *ft.*

The wetted perimeter at Max. High Flood,

$P_1 = P + 2\ (r_1 - r)$

$= 400 + 2{\cdot}90 = 403$ *ft. approx.*

The waterway area at Max. High Flood

$A_1 = P_1 \times r_1$

$= 403 \times 8{\cdot}95 = 3{,}610$ *sq. ft. approx.*

The probable maximum velocity

$$V_1 = V \times \left(\frac{Q_1}{Q}\right)^{\frac{2}{5}}$$

$= V \times (1\frac{1}{3})^{\frac{2}{5}}$

$= 1{\cdot}12\ V.$

$= 1{\cdot}12 \times 13{\cdot}7 = 15{\cdot}33$ *ft. /sec.*

The maximum discharge

$Q_1 = A_1\ V_1$

$= 3{,}610 \times 15{\cdot}33$ = *55,300 cusecs approx.* which is near enough to 55,000 cusecs to be assumed as correct.

∴ R. L. of Max. H. F. (without afflux)

$= O.\ H.\ F.\ L. + (r_1 - r) = 131{\cdot}80 + 1{\cdot}45 = 133{\cdot}25.$

It is proposed to provide a waterway B of 320 ft. clear between piers and abutments, by 4 spans of 80 ft. clear width.

From cross section, area of clear waterway at Max. H. F.

$A_2 = 3{,}100$ *sq. ft.*

and the average depth.

$$d = \frac{A_2}{B} = \frac{3{\cdot}100}{320} = 9{\cdot}7 \textit{ ft.}$$

The average discharge per foot width of clear opening

$$q=\frac{Q_1}{B}$$

$$=\frac{55,000}{320}=\mathit{172\ cusecs.}$$

The afflux

$$h=H-\cdot0155\ V_1^2$$

where $H^{\frac{1}{2}}(H+1\frac{1}{2}\ d)=\frac{q}{5\cdot35c}$

C=·80 for round-nosed piers

$$\therefore H^{\frac{1}{2}}(H+1\frac{1}{2}\ d)=\frac{172}{5\cdot35\times\cdot8}$$

or $H^{\frac{1}{2}}\ (H+14\cdot55)=40\cdot3$

By trial and error,

$H=\mathit{4\cdot5}$

$$\therefore h=H-\cdot0155\ V_1^2$$

$$=4\cdot5\times(\cdot0155\times15\cdot33^2)=\mathit{\cdot84\ ft.}$$

The velocity under the bridge

$$V_2=\frac{1\cdot1\times A_1\ V_1}{A}$$

Where $A_2=(d+\frac{h}{2})\ B$

$$=(9\cdot7+\cdot42)\ 320$$

$$=10\cdot12\times320=\mathit{3,238\ sq.\ ft.}$$

$$\therefore V_2=\frac{1\cdot1\times3,610\times15\cdot33}{3,238}$$

$$=\mathit{18\cdot8\ ft/sec.}$$

The waterway is slightly contracted only, and the current is straight.

∴ the maximum probable depth to scour below Max. H. F. L.

$$D_1=\frac{1\cdot3\ V_2}{V}\text{ or }\frac{2\cdot1\ r\ V_2}{V}$$

whichever is greater.

$$\therefore D_1=\frac{1\cdot3\times12\cdot5\times18\cdot8}{13\cdot7}\text{ or }\frac{2\cdot1\times7\cdot5\times18\cdot8}{13\cdot7}$$

$$=22\cdot3\text{ ft. or }21\cdot7\text{ ft.}$$

Take $D_1=\mathit{22\cdot3\ ft.}$

Depth of foundations below Max. H. F. L.

$$D_2=1\frac{1}{3}\times D_1$$

$$=1\frac{1}{3}\times22\cdot3=\mathit{29\cdot7\ ft.}$$

Depth below average bed level

$$=1\frac{1}{3}\ D_1-d=29\cdot7-9\cdot7=\mathit{20\ ft.}$$

R. L. of mean bed level being 124·30

R. L. of foundations will be *104·30.*

The clearance of the superstructure above Observed H. F. L.

$$C=\left(\frac{Q_1}{Q}\right)^{\frac{2}{3}}r-r$$

subject to a minimum of $\frac{3}{4}$ r = 5·6 ft.

Make 6 *ft. i. e.* R. L. 131·80+6 = *137·8.*

No extra allowance for traffic is necessary, as the stream is not navigable.

Note.—The approximate rule (Chapter V, equation 29 a) gives depth of foundations below mean bed level.

= 1·6 D or 2·5 r

= (1·6 × 12·5) or (2·5 × 7·5).

= 20 ft. or 18·7 ft.

i. e. *20 ft.*

This agrees with the depth 20 ft. calculated by the complete method.

EXAMPLE 2.

6′ Reinforced Concrete Slab Culvert.

See Plate XII and Chapter VI, para. 14.

Reinforced concrete slab with road metal directly on top of the slab.

Effective span = about 6′ + depth of slab

= 6′+6″ = 6½′

Dead load from metal 70 lbs./sq. ft.

From Table XII, or Plate XXII, a 8½″ slab would do, and from Table XIV this requires ⅝″ bars at 6½″ centres with the centre of the bars 1¼″ above the bottom of the slab.

The following details illustrate the method of calculation.

Loads and Moments on R. C. Slab.

From Plate XI, Case I for longitudinal slabs, the maximum live load bending moment from the rear wheels at the centre of the span

$$=M=\frac{PL}{4}=\frac{22,400\times6\frac{1}{2}'}{4}=36,300 \text{ ft. lbs.}$$

and for one wheel $M=\frac{36,300}{2}=18,150$ ft. lbs.

The effective width of slab across the slab span over which this may be considered as distributed is (see Chapter V, para. 31).

e = $\frac{2}{3}$ L+C with a maximum of 5 ft.

C = width of tyre = 1·32.

∴ e = ($\frac{2}{3}$ x 6½) 1·32 = 4·33   1·32 = 5·65 ft.

Make e = 5 ft.

Then the live load moment per foot width of slab

$$\frac{M}{e}=\frac{18\cdot150}{5}=3,630 \text{ ft. lbs.}$$

Dead Load of 6″ metal 1½″ earth cushion = 70 lbs. / sq. ft.
8¼″ slab at 150 lbs. per cubic foot = 103 ,,
Total Dead Load . . . . = 173 ,,

Dead Load Moment per ft. width of slab

$$=\frac{WL}{8}=\frac{173\times 6\frac{1}{2}'\times 6\frac{1}{2}}{8}=915 \text{ ft. lbs.}$$

∴ Total Moment per foot width of slab

$$= L\,L\,M+D\,L\,M = 3{,}630+915 = 4{,}545 \text{ ft. lbs}$$

[See Volume I, M. E. S. Handbook]

Design of Slab. From Plate XXXI for stresses of 16,000 and 600 lbs /sq. inch in the steel and concrete

$$\text{Steel ratio } p = \cdot 0067,\quad R=\frac{M}{bd^2}=95,$$

$$\text{and } d=\sqrt{\frac{M}{R\,b}},=\sqrt{\frac{M}{95\,b}} \quad \text{where M is in in. lbs.}$$

$$\text{Hence depth of slab } d=\sqrt{\frac{4{,}545\times 12}{95\times 12}}=6\cdot 92''.$$

Adding 1¼″ to cover the steel, total thickness

h = 6·92+1·25 = 8·17″. Say 8¼″.

∴ Area of steel required pbd = ·0067 × 12″ × 6·92″

= ·57 sq. in. per ft. width.

From Table XXXVII, this requires ⅝″ diameter bars at 6½″ centres.

Shear need not be investigated, as the load is much less than 2120 lbs./sq. ft. (See Table XXXVIII). Alternate bars will be bent (see Plate XXXV) and placed along the span with their centres 1¼″ from the underside of the 8¼″ slab. The ends of all bars will be hooked, and if splices are necessary they must be made by interlocking hooks made on the ends of the bars.

½″ diameter distributing bars should be placed across the span, and over the bars along the span, at 2½′ to 3′ centres.

Bearings. From Plate XXII, the length of bearing on the abutment

$$=K=\tfrac{1}{2}'+\frac{S}{40}=\tfrac{1}{2}'+\frac{6}{40}=\cdot 65'=8''$$

$$\text{Thickness of bedstone}=\frac{K}{2}=4''.$$

Camber of Slab. The slab must be cambered during construction to $\frac{S}{240}=\frac{6'}{240}=\cdot 3''$, say $\frac{5}{16}''$.

Notes. *Vide* Plate XXII, this slab can safely carry a superimposed fill of $H=\frac{60}{\text{span}}=\frac{60}{6}\times 10$ ft.

If the fill were 15 feet, total thickness of slab

$$h = \frac{S}{2 \cdot 3} \sqrt{H} = \frac{6}{2 \cdot 3} \sqrt{15} = 10''.$$

From Table XIV this requires ⅝" bars at 5" centres.

Longitudinal slabs similar to the above may also occur between the cross floorbeams of a plate girder or truss bridge. If continuous the moments are multiplied by ⅔ for fully continuous slabs, or by ⅘ for slabs with one end continuous and one end simply supported.

Abutments.

The bed level of the culvert is 6′ below the surface of the road, and the foundations are 3 ft. deep.

From Plate XVII, Type I, for masonry in lime

a = 18″.

The thickness of the abutment at ground level is ·40h = ·40 × 6′ = 2·4.′ Say 2′ 6.″

The projection of the toe $= \frac{H}{10} = \frac{9}{10} = \cdot 9'$. Say 1′.

The depth of the toe projection from its junction with the vertical face must not be less than 2 ft.

The total base width A = 1′+2′ 6″=3′ 6,″ which is greater than ·35 H, and is therefore satisfactory.

If scour is anticipated, cut-off walls should be provided, as shown on Plate XV. The foundations in this case will be only 18″ or 2′ deep, and the floor will be paved with stone in cement or good lime mortar.

Example 3.

6′ steel troughing span.

See Plate XXI and Chapter VI, para. 11.

Although steel troughing is very uneconomical in the case of such a small span, a 6′ culvert as in Example 2 will be considered to illustrate the method

Loads and Moments on ecking.

The dead load is practically the same as in Example 2, and the live load is exactly the same.

∴ Total moment per ft. width of decking

= 4,545 ft. lbs. as in Example 2.

Design of steel troughing.

Section modulus $I/Y = Z = \frac{M}{f}$, where M is in inch lbs., and f = 6½ tons/sq. in. for steel troughing.

$$\therefore Z = \frac{4{,}545 \times 12}{6\tfrac{1}{2} \times 2{,}240} = 3 \cdot 75$$

From Table XX, for pressed steel troughing of

section 4″ × $\frac{5}{16}$″, — Z = 6·0,

or for 5″ × $\frac{5}{16}$″, — Z = 6·8.

This is amply strong.

Though flooring is equivalent to covering the floor with a $\frac{5}{16}''$ or $\frac{3}{8}''$ plate, the deeper the section the greater being its strength. The weight per square foot, however, obviously remains almost constant, and there is therefore practically no economy between a $4'' \times \frac{3}{8}''$ and a $9'' \times \frac{3}{8}''$ or $12'' \times \frac{3}{8}''$ section. For this reason troughing should only be used in deep sections and for long spans.

Steel Troughing is an expensive type of flooring, and should generally not be used for spans of less than 10 or 12 ft.

Plate XXI gives full details for troughing.

Abutments.

The design of the abutments is exactly the same as for Example 2.

Example 4.

Reinforced concrete slab decking continuous over longitudinal Reinforced concrete beams.

See plate XXIII and Chapter VI, para. 15, also Volume I, Chapter VIII.

Dead Load from metal = 70 lbs. /sq. ft.

Design of R. C. decking.

For 18′ roadway, span centre to centre of beams = 7′3″, and the beams will be 1′4″ wide plus 2″ batter.

∴ Clear span of slab = 7′3″ — 1′6″ = 5′9″

Effective span = 5′ 9″ depth of slab = 5′9″+6″ = 6′3″.

From Table XIII, a $6\frac{1}{2}''$ slab will be suitable.

From Table XIV, this requires $\frac{1}{2}''$ diameter bars at $5\frac{1}{2}''$ centres.

The following details illustrate the method of calculation.

Loads and Moments on decking.

From Plate XI, Case I, for cross slabs, the maximum live load bending moment from the rear wheels at the centre of the span

$$M = \frac{P}{4} \times \left(\frac{L}{2} - \frac{1{\cdot}32}{4} \right) = \frac{22{,}400}{4} \times \left(\frac{6{\cdot}25}{2} - \frac{1{\cdot}32}{4} \right)$$

$$= 5{,}600 \times 2{\cdot}8 = 15{,}700 \text{ ft. lbs.}$$

From Chapter V, para. 31, the effective width of slab across the slab span over which this is distributed

$$e = \tfrac{2}{3}\,(L+C) = \tfrac{2}{3}\,(6{\cdot}25+1{\cdot}32) = 5{\cdot}05 \text{ ft}$$

Live load moment per ft. width of slab

$$= \frac{M}{e} = \frac{15{,}700}{5{\cdot}05} = 3{,}100 \text{ ft. lbs.}$$

Dead Load of 6″ metal plus $1\frac{1}{2}''$ earth = 70 lbs. /sq. ft.

Dead Load of $6\frac{1}{2}''$ slab at 150 lbs./c. ft. = 80 „

∴ Total Dead Load................150 lbs. /sq. ft.

∴ Dead Load Moment per ft. width of slab

$$= \frac{WL}{8} = \frac{150 \times 6{\cdot}42 \times 6{\cdot}42}{8} = 775 \text{ ft. lbs.}$$

∴ Total Moment per ft. width

= L L M+D L M = 3,100+775 = 3,875 ft. lbs.

As the slab is fully continuous over the supports, the effective moment is $\frac{2}{3}$ of the above, *i.e.*, 3,875 x $\frac{2}{3}$ = 2580 ft. lbs. (or 30,960 in. lbs.)

Design of decking.

From Plate XXXI, as in Example 2

$$\text{Depth of slab } d = \sqrt{\frac{M}{95 \times b}} = \sqrt{\frac{2.580 \times 12}{95 \times 12}} = 5{\cdot}20''$$

Adding 1" to cover the steel, total thickness
h = 6·20" Say $6\frac{1}{4}$".

Area of steel required = pbd = ·0067 × 12" × 5·20"
= ·42 sq. in. per ft. width.

From Table XXXVII, this requires $\frac{1}{2}$" diameter bars at $5\frac{1}{4}$ centres.

As the slab is continuous, this amount of steel must be provided at the centre of the span and also over the beams. Plate XXIII illustrates how this is done.

Bends in bars are made as shown on Plate XXXV.

EXAMPLE 5.

Reinforced concrete Beam Bridge with 40' spans.

See Plate XXIII and Chapter VI, para. 15, also Volume I, Chapter VIII

A nullah requires a bridge of 160 ft clear opening, 18' roadway.

The height from the surface of the road to the bed of the nullah is 15 ft., and the foundations are calculated to be safe against scour at 12 ft. below nullah bed level. ∴ total height = 27'0".

The total height of the piers should be about 22', as the depth of the beams plus the roadway thickness is about 5 ft.

From Chapter V, para. 6, Economic
span = $1\frac{3}{4}$ × 22' = $38\frac{1}{2}$'

4 spans of 40 ft. clear will be suitable.

R. C Design adopted.

The design will be according to Plate XXIII and Table XXIV, from which full details for making the working drawings can be obtained.

This 3 beams design has been chosen for its simplicity. A 2 beam design would require a much thicker slab and heavier and deeper beams, with a very large amount of steel to arrange in them. 4 or 5 beams would increase the cost of form-work, would complicate the bending of steel in the slab, and would require more concrete and steel in the main beams, as against a very trifling saving in concrete and steel in the slab. The 3 beam design gives the best arrangement, both for design and for construction.

The following details illustrate the method of calculation and design.

R. C. Slab. Slab thickness, as calculated in Example 4, = $6\frac{1}{4}$".

R. C. Beams. From Table XXIV, total depth H of beam = 4'4".

Width of beam Q = 1′4″ ; 1′6″ at slab.
Position of bars F = 3″, G = 3″.
Depth to centre of steel

$$d = H - F - \frac{G}{2} = 52'' - 3'' - 1\tfrac{1}{2}'' = 47\tfrac{1}{2}''.$$

Span centre to centre of bearings

$$= 40' + \left(\tfrac{1}{2}' + \frac{S}{40}\right) = 41\tfrac{1}{2}'.$$

The centre beam will first be investigated

From Plate XI, Case II, for longitudinal stringers or beams. **Loads and Moments on beams.**

$$\text{Live Load Moment} = (P + P_1) \times \frac{X^2}{L}$$

$$\text{and } X = \frac{L}{2} - 2{\cdot}04' = \frac{41\frac{1}{2}}{2} - 2{\cdot}04 = 18{\cdot}71'$$

$$\therefore L\ L\ M = (22{,}400 + 15{,}400) \times \frac{18{\cdot}71^2}{41\frac{1}{2}} = 3{,}19{,}000 \text{ ft. lbs.}$$

This is distributed laterally over a 10 ft. width (Chapter V, para. 31).

Hence on a 7′3″ width L L M on beam = 2,31,000 ft. lbs.
Dead load of 6″ metal + $1\frac{1}{2}$″ earth = 70 lbs. /sq. ft.
Dead load of $6\frac{1}{4}$″ slab at 150 lbs. /c. ft. = 78 lbs. /sq. ft.

∴ Total Dead Load 148 ,,

∴ Dead Load Moment

$$= \frac{WL}{8} = \frac{148 \times 7{\cdot}25 \times 41\frac{1}{2} \times 41\frac{1}{2}}{8}$$

= 2,31,000 ft. lbs.

Load due to weight of girder below slab
= 3′10″ × 1′5″ × 150 lbs. = 815 lbs./ lineal ft.

$$\text{Dead Load Moment} = \frac{WL}{8} = \frac{815 \times 41\frac{1}{2} \times 41\frac{1}{2}}{8} = 1{,}75{,}000 \text{ ft lbs.}$$

Total Moment = L L M + D L M
= 2,31,000 + (2,31,000 + 1,75,000) = 6,37,000 ft. lbs.

$$\text{Width of T beam flange } b = \frac{\text{span}}{4} = \frac{41\frac{1}{2}'}{4} = 10'4\tfrac{1}{2}''$$ **Design of beam.**

or = (12 × slab thickness) + b′
= (12 × $6\frac{1}{4}$″) + 18″ = 7′9″
or = distance centre to centre of beams = 7′3″

∴ Adopt 7′3″, which is the smallest.

See Plate XXXIII.

$$R = \frac{M}{bd^2} = \frac{6{,}37{,}000 \times 12}{87 \times 47\frac{1}{2} \times 47\frac{1}{2}} = 39 \text{ and } \frac{t}{d} = \frac{6\frac{1}{4}''}{47\frac{1}{2}''} = {\cdot}131,$$

∴ j = ·94 and compressive stress f_c = 400 lbs. /sq. in.

∴ area of reinforcement steel required

$$= \frac{M}{f_s \cdot j \cdot d} = \frac{6{,}37{,}000 \times 12}{16{,}000 \times {\cdot}94 \times 47\frac{1}{2}} = 10{\cdot}35 \text{ sq. in.}$$

From Table XXXVI, adopt 6 bars of $1\frac{1}{4}''$ diameter

$=6\times1{\cdot}227=7{\cdot}362$ sq. in.

$+2$ bars of $1\frac{3}{8}''$ dia. $=2\times1{\cdot}485=2{\cdot}970$ sq. in.

Total 10·332 sq. in.

Width of beam $=$ (3 $\times$ dia. of bars) $+$ 4" $=$ 8", but should not be less than $\frac{1}{3}$ of depth. Make 16".

The bars are laid in 2 layers with an interval of 3", the top layer consisting of 4 — $1\frac{1}{4}''$ bars, and the bottom layer of 2 — $1\frac{1}{4}''$ bars and 2 — $1\frac{3}{8}''$ bars.

3" of concrete will be given above and below, to cover the bar and the stirrups (see below).

The bars must be securely wired in position, with spacing blocks of cement concrete between the 2 layers and the forms. In the top layer 2 bars will be bent up at $\cdot25\times41\frac{1}{2}'=10'4''$ and 2 at $\cdot15\times41\frac{1}{2}'$ $=6'3''$ from the centre of the bearing, or, (adding half the bearing length) 11'1" and 7'0" from the end of the beam, respectively.

Using Table XXIV and Plate XXIII, the above details are obtainable without calculation.

Notes. *N B.*—It should be noted in regard to the design of the T beam, that it might be made much shallower if this were not precluded for practical reasons, *viz.* — economising in steel, avoidance of the difficulties attendant upon placing large masses of bars in the beam, and the necessity of providing a sufficient area of concrete to resist shear.

Thus for a shallower beam, in which kd is equal to or less than t. using the results on Plate XXXI as in Example 2, $R=\frac{M}{bd^2}=95$,

whence d $=30''$, and area of steel $=16{\cdot}4$ sq. in., which would necessitate 8 bars of $1\frac{5}{8}''$ diameter.

The shear would then be about 134 lbs / sq. in., whereas 120 lbs./ sq. in. is the maximum shear permissible.

Shear. From Plate XI, Case II, for maximum reaction, Maximum Reaction of Live Load Shear from Road Roller

$$=P+\left(P_1\times\frac{L-a}{L}\right)$$

$$=22.400+\left(15{,}400\times\frac{41\frac{1}{2}-10}{41\frac{1}{2}}\right)=34{,}100 \text{ lbs.}$$

This is distributed laterally over 10 ft. (Chapter V. para. 31).

$$\therefore \text{Live Load Shear}=\frac{34{,}100\times7{\cdot}25'}{10'}=24{,}700 \text{ lbs.}$$

Dead Load $=(148\times7{\cdot}25')+815$ lbs.

$=1{,}885$ lbs./ lineal ft.

$$\text{Dead Load Shear}=\frac{W}{2}=\frac{1{,}885\times41\frac{1}{2}}{2}=39{,}200 \text{ lbs.}$$

$\therefore$ Total Shear $=$ L L S $+$ D L S.

$=24{,}700+39{,}200=63{,}900$ lbs.

$$\text{Shear per sq. in. } f_v = \frac{\text{shear}}{b' \times j \times d.}$$

$$= \frac{63{,}900}{17'' \times \cdot 94 \times 47\tfrac{1}{2}''}$$

$$= 85 \text{ lbs./ sq. in.}$$

As 60 lbs./ sq. in. is the maximum shear allowable in a beam unreinforced for shear, this beam will require reinforcement. **Shear Reinforcement.**

Assuming that the concrete carries $\frac{1}{8}$ and the bent bars $\frac{1}{8}$ of the shear, half of the shear is left to be carried by stirrups. This assumption is very much on the safe side, but it is adopted for the purpose of getting practical diameters for the stirrup bars, which, besides carrying the shear, have to connect the flange to the stem of the T beam.

s = spacing of stirrups in inches

Tensile stress on the steel f_s = 12,000 lbs./ sq. in.

Area of one leg of stirrup

$$= a = \frac{f_v \times b' \times s}{f_s} = \frac{85 \times 17'' \times 10''}{4 \times 12{,}000} = \cdot 30 \text{ sq. in.}$$

Using Table XXXVII, $\frac{5}{8}''$ stirrups at 10″ centres are required.

This spacing will be continued from the end of the girder to $\frac{L}{4} = \frac{41\frac{1}{2}'}{4} = 10'$—$4\frac{1}{2}''$ from the centre; from this point the spacing may be 20″ to the centre of the span, as the shear decreases from a maximum at the bearing to nearly zero at the centre, under a moving load.

The stirrups are bent U shaped, with their ends hooked over two $\frac{1}{2}''$ diameter bars, and are wired to the main reinforcing bars.

The camber to be given to the beams **Camber of beams.**

$$= \frac{S}{240} = \frac{40' \times 12}{240} = 2''$$

Using Table XXIV and Plate XXIII, the above details are obtainable without calculation.

Thickness of cement concrete bedstone **Bearings.**

$$= \tfrac{2}{3} K = \tfrac{2}{3} \times 1'6'' = 1'0''.$$

Two $\frac{1}{4}''$ bearing plates will be provided, each plate securely fastened, one to the girder and one to the bedstone, by $\frac{5}{8}''$ diameter counter-sunk bolts, the plates to be finished smooth on the sliding surfaces.

A vertical end slab J, 9″ thick, will be provided, reinforced as shown on Plate XXIII. This should be moulded with the beams, in order to retain the bank behind and brace the beams. The under surface of this end slab will be 1″ clear of the bedstone. **End Slab.**

The abutments will be of lime masonry, with face vertical, according to Plate XVII, Type I. **Abutments.**

Beam seat a $= 32''$.

Thickness of bedstone $= X = \frac{1}{3} + \frac{S}{60} = \frac{1}{3} + \frac{40'}{60} = 1$ ft.

Height from ground level to surface of road
$h = 15$ ft.

Thickness of abutment at ground level
$= \cdot 40\ h = \cdot 40 \times 15 = 6$ ft.

Depth of foundations $= 12$ ft.

Total height of abutment $H = 27$ ft.

Projection of toe $= \frac{H}{10} = \frac{27'}{10} = 2\cdot7'$ Say $2'9''$.

Hence total width of base $= 6' + 2'9'' = 8'9''$. As this is about $= \cdot 35\ H$ it is satisfactory.

The height at which the toe joins the vertical face must not be less than $2'9'' \times 2 = 5'6''$.

To find the approximate pressure on the foundation at the toe :—

$$\frac{\text{Projection of toe}}{\cdot 40\ H} = \frac{2'9''}{\cdot 40 \times 27'} = \cdot 255 = \frac{1}{4}\ i.e.\ T = \frac{B}{4}$$

From Plate VI, for $T = \frac{B}{4}$ $f = \cdot 48$ p.

$\therefore$ toe pressure $f = \cdot 48 \times 230\ H$
$= \cdot 48 \times 230 \times 27 = 3{,}000$ lbs./ sq. ft.

The toe pressure will probably be increased to about 3,500 lbs./ sq. ft. by the weight of the bridge. As the foundation material can take 2 tons = 4,480 lbs / sq. ft., the pressures are safe.

The pressures can be accurately calculated by the method illustrated in Chapter IV, para. 11.

The abutments will be arranged with 45° splayed wing walls as shown on Plate XVII. The wing walls will be designed to fit the abutment in accordance with Chapter VI, para. 6.

Piers. The piers will be of cement masonry in accordance with Plate XVIII, Type I.

Cap thickness $= \frac{1}{3} + \frac{S}{60} = \frac{1}{3} + \frac{'}{60} = 1'$.

$t = \frac{3}{4}' + \frac{S}{25} = \frac{3}{4}' + \frac{40}{25} = 2\cdot35'$

Width of bearing $K = 1'6''$.

Hence top width of pier for
$(2K + 1'')$ clearance $= 3'1''$.

This thickness will do for a height of
$6 \times 3'\ 1'' = 18'6''$.

Below this level the pier will be stepped out to suit the width of foundation necessary at safe pressures. The width of the base will be approximately.

$\frac{H}{3} = \frac{22'}{3} = 7'4''$

Load on Pier Foundations =

Live Load as for Shear. . . . . . =34,100 lbs.
+Dead Load from 6" metal and slab, say 20' wide to allow for kerb = 20' (width)×43' (length) × 150 lbs./sq. ft. . . . =1,29,000 lbs.
+Weight of beams = 3×43'×815 lbs./ ft. run . . . =1,05,000 lbs.

Load from Bridge . . . . =2,68,100 lbs.
+weight of Pier (approximate) . . . =3,60,000 lbs.
Total load . . =6,28,100 lbs.

Area of base required at 4,480 lbs./ sq. ft.

$= \frac{6,28,100}{4,480} = 140$ sq. ft.

A base 5'6" wide by about 26' long is sufficient.

The straight part of the piers will be 15'6" long, with cutwaters beyond, as shown on Plate XVIII.

The beams, kerb, and handrail must be free to move under temperature changes of length, and must not interfere with the abutments or adjoining parts; and allowance of 1" movement for every 100 ft. of adjoining span is sufficient.

EXAMPLE 6.

40' R. S. Joist spans with reinforced concrete flooring.

See Plate XIX and Chapter VI, para. 10.

Roadway 18' wide.
6" metal, on 1½" earth cushion.
Take the same data as in Example 5.
Height of Pier will be about 24'.
Economic span = 1¾ × 24' =42' (Chapter V, para 6)

4 spans of 40 ft. clear to provide 160 ft. clear opening, will be suitable.

The design will be according to Plate XIX, from which full details for making the working drawings can be obtained.

In this design the 7 R.S. Joists are at 3'1" centres; they support a 6¼" slab designed and reinforced as simply supported. This arrangement requires no bending of steel, at a cost of a slab 1" thicker than if designed and reinforced as continuous. R. C. Flooring.

The close spacing of the R.S. Joists should minimize any tendency of the slab to crack over the R. S. Joists, and as the deflection of the slab will be very small, and the load is applied through a

$7\frac{1}{2}''$ cushion of surfacing material, no trouble need be anticipated from this.

If 3 or 4 beams were used, the slab spans would require to be reinforced for continuity, necessitating extra labour for bending and placing the steel.

The cross floor slab should be designed in the same manner as in Example 4, with the difference that it is simply supported.

The details can also be obtained direct from Table XIII.

From Table XIV, $\frac{1}{2}''$ diameter bars at $5\frac{1}{2}''$ centres, placed with their centres 1" above the bottom of the slab, are required for the $6\frac{1}{4}''$ R.C. Slab.

R. S. Joists.

$$\text{Effective span} = S+(\tfrac{1}{2}+\frac{S}{40}) = 41\tfrac{1}{2}'$$

Loads and Moments.

From Example 5, the live load moment for a 12 ton road roller on a $41\frac{1}{2}'$ span = 3,19,000 ft. lbs. (See also Table XII.)

This is assumed as uniformly distributed over a 10 ft. width of bridge (Chapter V, para. 31).

Hence the Live Load Moment for one R. S. Joist $= \frac{M}{10} \times$

$$\text{spacing} = \frac{3{,}19{,}000 \times 3{\cdot}08'}{10} = 98{,}250 \text{ ft. lbs.}$$

Dead load of slab and metal = 150 lbs./ sq. ft.

∴ Dead Load Moment for one R. S. Joist

$$= \frac{WL}{8} = \frac{150 \times 3{\cdot}08' \times 41\tfrac{1}{2} \times 41\tfrac{1}{2}}{8} = 99{,}700 \text{ ft. lbs.}$$

Dead Load Moment from weight of R. S. Joist (assume 75 lbs per lineal ft.)

$$= \frac{WL}{8} = \frac{75 \times 41\tfrac{1}{2} \times 41\tfrac{1}{2}}{8} = 16{,}200 \text{ ft. lbs.}$$

Total bending moment on one R. S. Joist

$= \text{L.L.M.} + \text{D.L.M.}$

$= 98{,}250 + (99{,}700 + 16{,}200) = 2{,}14{,}150$ ft. lbs.

Design of Joist.

$$\text{Section Modulus } \frac{I}{Y} = Z = \frac{M}{f} = \frac{2{,}14{,}150 \times 12}{7\tfrac{1}{2} \times 2{,}240} = 152.$$

A new British Standard Section R.S. Joist 22" × 7" @ 75 lbs. for which Z = 152, will do.

Bearings.

$$\text{Length of Joist on bearing} = K = \tfrac{1}{2} + \frac{S}{40} = 1\tfrac{1}{2}'$$

Thickness of bedstone $= \frac{4}{9}$ K = 8".

The joists will rest on $\frac{1}{2}''$ bearing plates on the bedstone. The outside joists will have one anchor bolt in each end, with the hole slotted in the joist flange. The spaces on the abutments between the joists will be filled with cement masonry laid on tar paper, to allow them to move with the joists.

A $2\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{1}{4}''$ angle iron spacer will be provided at the centre of the span, tying the bottom flanges together. **Spacer.**

The reinforcement bars are laid directly on the R.S. Joists flanges. Alternate bars are turned up into the kerb, and every 7th bar has both its ends hooked around the flange of the joists to prevent them from spreading. The bottom of the slab is $\frac{3}{4}''$ below the top of the R.S. Joist flange. **R. C. Slab Reinforcement.**

The design of the abutments and piers is the same as in Example 5. The straight part of the piers will be 18′ long, with the cutwaters projecting beyond. **Abutments and Piers.**

EXAMPLE 7.

R S. Joist Splice.

See Plate XIX and Chapter VI, para. 13.

A splice for a $24'' \times 7\frac{1}{2}''$ @ 100 lbs. R. S. Joist is to be designed. Section Modulus Z =221.

b = width of flanges in inches.
d = depth of joist.
t = thickness of web of the joist in inches.
r = resistance of one rivet in double shear or in bearing in the flange.

Thickness of plate on each side of flange

$$= \frac{5 \times Z}{8 \times d \times b} = \frac{5 \times 221}{8 \times 24'' \times 7\frac{1}{2}''} = \cdot 77''.$$

Use $\frac{3}{4}''$ plates and make the top plate 8″ wide.

Thickness of plate on each side of web

$$= \frac{5t}{8} = \frac{5}{8} \times \cdot 6'' = \cdot 375'' = \frac{3}{8}''.$$

Number of rivets in each flange on each side of splice=

$$\frac{7\frac{1}{2}\,Z}{d \times r} = \frac{7\frac{1}{2} \times 221}{24 \times 4\cdot 81} = 14\cdot 5. \text{ Make 16.}$$

r= 4·81 tons for a $\frac{7}{8}''$ diameter rivet handriveted. The pitch is $3\frac{1}{2}''$ for $\frac{7}{8}''$ rivets, *i.e.*, total pitch $= 4 \times \frac{7}{8}'' = 3\frac{1}{2}''$.

∴ total length of flange splice plates

$=16 \times 3\frac{1}{2}'' = 4'8''$.

The web plates must have 2 vertical lines of $\frac{7}{8}''$ rivets at $3\frac{1}{2}''$ pitch, on each side of the splice.

EXAMPLE 8.

Masonry arch Bridge with 25′ spans.

See Plate XVI and Chapter VI, para. 3.

18′ roadway. Hammer dressed voussoir stone in lime arches. A clear opening of 100 lineal feet is required.

The height is limited, therefore an arch of low rise will be required. Make Rise $=\frac{\text{Span.}}{5}$

The height from the surface of the road to high flood water level is 8′10″, to average bed level 14′6″, and to the bottom of the foundations 20′6″. The height from the crown of the arch to the foundations is about 17′.

For economy, the span should be greater than 17′, and should be as large as the headroom will admit.

When drawn to scale 4 spans of 25′ seem the most suitable.

6 spans of 16′8″ or 5 spans of 20′ would introduce extra piers, and would cost more for excavation and pier masonry, besides causing greater obstruction to the stream

As the headroom is limited, the clearance of the arch above observed high flood level should be the minimum allowable.

Distance of water level from springing

$$X = \tfrac{1}{2}\ (\text{average depth} - \text{rise of arch})$$
$$= \tfrac{1}{2}\ (5{\cdot}66' - 5') = +\ {\cdot}33'$$

As X is plus, the springing should be 4″ above O.H.F L. or $(5{\cdot}66+{\cdot}33) = 6'$ above average bed level.

All the information necessary for making the working drawings is given on Plate XVI, but to illustrate the equations they will be worked out.

Arch Ring. Radius of arch ring

$$R = \frac{a^2+r^2}{2r} \qquad \text{(Plate XVI, Equation 2)}$$

$$\frac{12\tfrac{1}{2}+5^2}{2\times 5} = 18{\cdot}125' = 18'\ \ 1\tfrac{1}{2}''$$

Basic thickness or depth of arch ring.

$$C = \sqrt{\frac{R+a}{4}} + 0{\cdot}2 \qquad \text{(Plate VI, Equation 3)}$$

$$\sqrt{\frac{18{\cdot}125+12{\cdot}5}{4}} + 0{\cdot}2 = 1{\cdot}59'$$

This must be multiplied by $1{\cdot}\tfrac{1}{3}$ for Hammer Dressed Voussoir stone in lime (see item 15 of Table on Plate XVI).

$$\therefore C = (1{\cdot}59 \times 1{\cdot}\tfrac{1}{3}) = 2{\cdot}12'.\ \text{Say } 2'\ 1\tfrac{1}{2}''.$$

Abutments. Thickness of abutment at springing

$$E = \frac{R}{5} + \frac{r}{10} + 2' \quad . \quad . \text{(Plate XVI, Equation 3)}$$

$$\frac{18{\cdot}125}{5} + \frac{5'}{10} + 2' = 6'2''$$

Back batter on abutment $n = \frac{24\,r}{S} = \frac{24\times 5'}{25'} = 4{\cdot}8$, *i.e.*, batter $= 4{\cdot}\tfrac{4}{5}$ or $2\tfrac{1}{2}$ in. horizontal per ft. vertical.

Thickness of footing, which will be of lime concrete $= 3'$. (Item 22, Plate XVI.)

As the springing is practically at water level in this case, the height of the abutment above the footing is $12'-3' = 9'$. As this is not more than $1\frac{1}{2}$ E $= 9'3''$, E does not require to be increased.

$$D = \frac{r+C}{2} \text{ (Plate XVI, Equation 6)}$$

$$= \frac{5'+2'2''}{2} = 3'7''.$$

The load on the abutment per lineal ft.
$= 17$ tons (item 24, Plate XVI).

For $H = S$,
Width of abutment

$$= 6'2'' + \frac{H}{4\cdot8} + 1'6'' \text{ for footing projections}$$

$$= 6'2'' + \frac{9'}{4\cdot8} + 1'6'' = 9'7''.$$

$$\text{Load per sq ft.} = \frac{17 \text{ tons}}{9'7''} = 1\cdot78 \text{ tons/ sq. ft}$$

But $H = S - 8'$
and the weight of 8 ft. of masonry

$$= \frac{8 \times 150}{2,240} = \cdot54 \text{ tons/ sq. ft.}$$

$\therefore$ average load per sq. ft. $= 1\cdot78 - \cdot54$
$= 1\cdot24$ tons/ sq ft.

This is safe as the permissible pressure is $2\frac{1}{2}$ tons/sq ft in this case.

Top thickness of pier $= 3'6''$ (Item 19, Plate XVI). Piers.

$$\text{Bottom width of base} = \frac{\text{Total height of Pier}}{2} \text{ approximate}$$

$$= \frac{12'}{2} = 6 \text{ ft.}$$

Height of pier above the bottom of the $3'$ thick footing $= 9$ ft.
This is less than 3 times the top thickness of $3'6''$ ($= 10'6''$).
$\therefore$ the footings need not be carried up higher in steps, as would be necessary otherwise.

For $H = S$.

Load on pier foundation per lineal ft. $= 21$ tons (item 23, Plate XVI).

Load on whole pier $= 21$ tons $\times$ $20'$ width $= 420$ tons.
Approximate area of pier foundation
$= (6' \times 29') = 174$ sq. ft.

$$\therefore \text{load per sq. ft.} = \frac{420}{174} = 2\cdot41 \text{ tons/sq. ft.}$$

This would reduce to about 2·2 tons/sq. ft. when the 8 ft. height of pier is deducted, which is safe for a permissible load of $2\frac{1}{2}$ tons/sq. ft.

Notes.—These calculations for loads on abutment and pier foundations are very approximate, but they indicate quickly whether the foundation pressures are safe or not. If the pressures do not thus appear safe, accurate calculations should be made, and the footings enlarged to give safe pressures on the foundation material.

Example 9.

Reinforced concrete open spandrel arch bridge with 100 foot spans.

See Plates XXVII and XXVIII, also Chapter VI, paras. 22-37, and Volume 1, Chapter VIII.

Roadway 18′ wide, metalled.

P.C. Concrete 1-2-4 ; lower part of piers only may be $1\text{-}2\frac{1}{2}\text{-}5$.

All steel to British Standard Specifications.

Live Load :- For Floor and Spandrels — 12 ton road roller + 25 per cent. impact.

Live Load :—For Arch Ribs — 140 lbs./ sq. ft. of roadway (Chapter V, para. 31).

Rise of arch = 25 ft.

2 arch ribs at 13 ft. centres will be used per span 12 spandrel panels each 8′ 7″ wide will be used per span.

Note.—Sufficient details are given in this Example to illustrate the special method of design and the methods of calculating and designing such reinforced concrete bridge work. Details not worked out here should be calculated and designed by the methods described in the relevant parts of this Handbook. Plate XXVIII gives all necessary working details for practical use, to be amplified and adapted as necessary to suit individual cases.

Flooring.

Floor Slabs. The slabs are semi-continuous at expansion joints, elsewhere continuous. For simplicity in design, the flooring will be taken as semi-continuous throughout.

Clear span = 8′7″ - width of floor beam

= 8′7″ — 1′ = 7′7″.

From Table XII, by interpolation, for a 7′7″ span thickness of slab

= $8\frac{3}{4}$″ (simple *i.e.* non-continuous) or $7\frac{1}{2}$″ (continuous)

= $8\frac{1}{4}$″ (semi-continuous).

From Table XIV, reinforcement bars of $\frac{5}{8}$″ diameter at $6\frac{1}{2}$″ centres will be suitable. The bars will be bent up as shown in Plate XXXV. For temperature reinforcement, $\frac{3}{8}$″ bars at 2′2″ centres will be given, across the slab.

Floor beams.

Floor Beams. Interval = 8′7″

Spans :—Centre span 13′

Roadway cantilever spans each $2\frac{1}{2}'$ at the two ends.

Dead load per lineal ft. of span **Loads and Moments.**

$$=\text{wt. of metalling} \quad . \quad . \quad . \quad . \quad 70 \times 8\frac{7'}{12}$$

$$+\text{wt. of floor slabs} \quad . \quad . \quad . \quad . \quad \frac{8\frac{1}{4}'' \times 150}{12} \times 8\frac{7'}{12}$$

$=600+885=1,685$ lbs.

Normal D L M on centre span

$$= \frac{WL^2}{8} = \frac{1,685 \times 13 \times 13}{8} = 35,600 \text{ ft lbs.}$$

D L M on cantilever (minus)

$$= \text{for roadway } \frac{WL}{2} = \frac{1,685 \times 2\frac{1}{2} \times 2\frac{1}{2}}{2} = 5,270 \text{ ft. lbs}$$

+ for handrail, wheel guard, and cantilever

$$WL = (400 \times 8\frac{7'}{12}) \times 3' = 10,300$$

$$= 5,270 + 10,300 = 15,570 \text{ ft lbs.}$$

$\therefore$ resultant D L M on centre span

$$= 35,600 - 15,570 = 20,030 \text{ ft lbs}$$

From Table XIII, live load per lineal ft. of floor beam $= 2,240$ lbs.

$\therefore$ L L M on centre span

$$= \frac{WL}{8} = \frac{2,240 \times 13 \times 13}{8} = 47,400 \text{ ft. lbs}$$

and L L M on cantilever (minus)

$$= \frac{WL}{2} = \frac{2,240 \times 2\frac{1}{2} \times 2\frac{1}{2}}{2} = 7,000 \text{ ft. lbs.}$$

$\therefore$ total moment on centre span

$$M_1 = \text{D L M} + \text{L L M}$$

$$= 20,030 + 47,400 = 67,430 \text{ ft lbs.}$$

and total moment on cantilever

$$M_2 = \text{D L M} + \text{L L M}$$

$$= -(15,570 + 7,000) = 22,570 \text{ ft lbs.}$$

End shear on centre span **Shear.**

$$V = \text{from dead load } \frac{W}{2} = \frac{1,685 \times 13}{2} = 10,950 \text{ lbs}$$

$$+\text{from live load } \frac{W}{2} = \frac{2,240 \times 13}{2} = 14,600 \text{ lbs}$$

$$= 25,550 \text{ lbs}$$

End shear on cantilever

V = dead load from roadway $= W = 1,685 \times 2\frac{1}{2} = 4,200$ lbs.

+dead load from handrail

and wheel guard $= W = 400 \times 8\frac{7}{12}$. $= 3,440$,,

+live load $= W = 2{,}240 \times 2\frac{1}{2}$ . . $= 5{,}600$ lbs.
$= 13{,}240$ lbs.

Design of Floor-beams. Centre span.

For the centre span of the floor beam :—
From the equation

Unit shearing stress $f_v = \dfrac{V}{j\, b'\, d}$ lbs./sq. in.

Minimum safe cross sectional area for the centre span, against an end shear of 25,550 lbs. at 120 lbs/sq. in. safe shearing stress.

$$b'd = \frac{V}{j \times f_v} = \frac{25{,}550}{\cdot 91 \times 120} = 232 \text{ sq. in.}$$

A suitable section will be

$b' = 11''$

$h = 24''$, so that $d = 24'' - 2'' = 22''$

and $b' \times d = 242$ sq. in.

This section will be economical, as the depth = about $\frac{1}{7}$ span

For the design of the T beam, see Plate XXXIII.

$$b = \frac{13'}{4} = 3'9'' \text{ (as this is the minimum).}$$

$$\frac{M}{bd^2} = \frac{67{,}430 \times 12}{39 \times 22 \times 22} = 43$$

$$\frac{t}{d} = \frac{8\frac{1}{4}}{22} = \cdot 37$$

∴ neutral axis is in slab, and from Plate XXXIII the unit compressive stress in the concrete

$f_c = 400$ lbs./sq. in. approx.

$j = \cdot 92$

Cross sectional area of reinforcement bars in centre span

$$A_s = \frac{M}{j\, d\, f_s} = \frac{67{,}430}{\cdot 92 \times 22 \times 16{,}000} = 2\cdot 49 \text{ sq. in.}$$

4 bars of $\frac{7}{8}''$ dia. will be suitable.

The bars will be bent up as shown on Plate XXXV.

Necessary width of beam

$= 4 \times (3 \times \frac{7}{8}) = 10\frac{1}{2}''$

∴ 11″ width will be suitable.

Stirrups.

As the shear stress = approx. 120 lbs./sq. in., U stirrups are necessary.

Assuming that the concrete and the bent up reinforcement bars carry half the shear,

End spacing of stirrups

$$= \frac{d}{4} = \text{Say } 6''.$$

With stirrups 12″ apart,

Tension in one stirrup

$$T = \frac{\frac{1}{2} V S}{j\, d} = \frac{\frac{1}{2} \times 25{,}550 \times 12''}{\cdot 92 \times 22''} = 7{,}600 \text{ lbs.}$$

Cross sectional area necessary for this $= \frac{7,600}{12,000} = \cdot 63$ sq. in., or ·32 sq. in. for one arm of the stirrup.

The spacing should be $\frac{1}{4}$ to $\frac{1}{3}$ depth of beam.

From Table XXXVII,

Stirrups of $\frac{1}{2}''$ dia. bar, at 7″ centres, will be suitable.

Stirrups will be given at this spacing on the outer thirds of the 13′ span; stirrups will be given in the centre third, at 12″ centres, for safety, although they are not required theoretically. Exact spacing according to the shear would save about 4 stirrups per beam in this case, which is not worth considering.

For the cantilever portion of the floor beams :— **Cantilever Span.**

$b = 11''$

$d = 22''$

$t = 8\frac{1}{4}''$.

$j = \cdot 88$ as for slabs, the tension being in the top and the compression in the bottom surface

End shear per sq. in.

$$f_v = \frac{V}{j\ b\ d} = \frac{13,240}{\cdot 88 \times 11 \times 22} = 62 \text{ lbs/sq. in.}$$

As the shear stress is over 60 lbs./sq. in.

2 U stirrups will be necessary.

For moment,

$$\frac{M}{bd^2} = \frac{22,570 \times 12}{11 \times 22 \times 22} = 42$$

$\therefore$ from Plate XXXIII, as $\frac{t}{d} = \cdot 37$

concrete stress $f_c = 350$ lbs./sq. in. approx.

Cross sectional area of reinforcement bars

$$A_s = \frac{M_2}{j\ d\ f_s} = \frac{22,570}{\cdot 88 \times 22 \times 16,000} = \cdot 88 \text{ sq. in.}$$

2 bars of $\frac{3}{4}''$ dia. will be suitable; these must be continued through the outer fourth of the centre span (*i.e.*, $\frac{13}{4} = 3\frac{1}{4}'$ in.).

The bent up pair of $\frac{7}{8}''$ reinforcement bars of the centre span will be stopped beyond the supports.

Expansion Joints.

See Plate XXVIII.

Expansion joints will be provided in the flooring system, at the abutments and piers, by giving double floor beams sliding on plates on the tops of the spandrel columns. Details are given on Plate XXVIII. These end floor beams are designed similarly **Expansion Joints.**

to the intermediate beams dealt with above, with the necessary modifications shown.

Spandrel Columns.

See Volume I, Chapter VIII.

Spandrel Columns.

Load on spandrel column.

=load from centre span of superimposed floor beam (equals end shear thereon)

+load from cantilever (ditto).

+load from arching between columns.

+wt. of spandrel column

=25 550 + 13 240 + (say) 900 + (max.) (18′ × 205)

=43,390 lbs.

Outside dimension of spandrel column should not be less than $\frac{1}{15}$ length of column

∴ Minimum cross section of end column (18′ long) = $\frac{18'}{15}$ = 14″ square

The remaining columns may be 12″ square, chamfered 3″ on the corners.

Concrete cross sectional area of 12″ column

=(12″ × 12″) - (3″ × 3″ × 4 ½) = 126 sq. in.

1 per cent. steel reinforcement = 1·26 sq. in.

4 reinforcement bars of ¾″ dia. will be suitable (area = 1·76 sq. in.).

¼″ binders will be given at 12″ centres.

The steel area is equivalent to a concrete area

A_s (n - 1) = 1·76 (15 - 1) = 24 6 sq. in.

∴ total equivalent concrete area of column = 126 + 24·6 = 150·6 sq. in. Say 150 sq. in.

For a safe load of 400 lbs/sq. in., a column of this size will carry 150 × 400 = 60,000 lbs.

The load being 43,390 lbs., 12″ × 12″ columns will be amply strong (end columns 14″ × 14″)

Columns over piers.

For architectural reasons, spandrel columns over piers will be of the following dimensions at the base, tapered up to the normal dimensions 14″ × 14″ :—

Width parallel to length of bridge = 2′.

(= the distance between the upper edges of the arch ribs at the springing).

Width across the bridge = 3′4″

(= the width of the arch rib).

ARCH RIBS.

A rough preliminary calculation indicates that ribs 22″ deep × 40″ wide at the crown and 36″ deep × 40″ wide at the springing will be suitable. **Arch Rib.**

Dead Load per rib, **Dead Load.**

At Crown

=wt. of 9′ width of metalling.

+wt. of $8\frac{1}{4}$″ floor slab

+wt. of handrail, wheelguard, and cantilever

+wt. of floor beam

+wt. of spandrel arching

+wt. of spandrel column

+wt. of arch rib

+wt. of 4 rib braces

$$=(9\times70)+(9\times103)+(\text{say})\ 400+\frac{1\times11\times15\frac{3}{4}\times150}{8\frac{7}{12}\times144}$$

$$=\frac{(\text{Say})\ 900}{8\frac{7}{12}}+0+\frac{22\times40\times150}{144}+\frac{4\times4\frac{10}{12}\times1\frac{1}{4}\times2\frac{1}{2}\times150}{100}$$

$$=630+927+400+147+105+0+920+91$$

=3,220 lbs./lineal ft.

Additional load at springing.

=for spandrel column (if at end)

+for increase in dimensions of arch rib

$$=\text{@ 185 lbs./sq. ft.}\ \frac{24\times185}{8\frac{7}{12}}$$

$$\frac{14\times40\times\sec\beta\times150}{144}$$

where β = inclination of springing line to horizontal.

$$=\tan^{-1}\frac{4\times\text{rise}}{\text{span}}\ \text{approx.}$$

$$=\frac{4\times25}{100}=1{\cdot}0.$$

$\therefore\ \beta=45^\circ$ and $\text{Sec}\ \beta=\text{Sec}\ 45^\circ=1{\cdot}41$

$\therefore$ additional load at springing

=(say) 520+583·33 Sec β or 583·33 Sec β.

=520+(583·33×1·41), or (583·33×1·41)

=approx. 1,340 or 820 lbs./lineal ft.

The maximum will be taken.

N.B.—In calculating the loads per ft. at the crown and springing the average dimensions, etc., throughout the span must be considered.

Live Load per rib @140 lbs./sq. ft. **Live Load.**

=9×140=1,260 lbs./lineal ft.

As this rib has a large rise, the rib axis will be laid out for dead load + $\frac{1}{2}$ live load. **Curve of Arch Axis.**

Hence (See Chapter VI, para. 30 and Plates XXVII and XXVIII), W_c will be taken as

$$=3{,}220+\frac{1{,}260}{2}=3850 \text{ lbs/lineal ft.}$$

and $W_s=3{,}220+1{,}340+\dfrac{1{,}260}{2}=5{,}190$ lbs/lineal ft

$$\therefore u=\frac{W_s}{W_c}=\frac{5{,}190}{3{,}850}=1.35 \text{ (Equation 2)}$$

$$y=\frac{4r}{L^2(u+5)}\left[6x^2+(u-1)\frac{4x^4}{L^2}\right] \text{ (Equation 1).}$$

$$\tan\beta=\frac{4r}{L^2(u+5)}\left[6L+2L(u-1)\right] \text{ (Equation 3)}$$

$$\text{Hence } \beta=\tan^{-1}=\frac{4\times25}{100\times100\times6{\cdot}35}(600+70)$$

$$=\tan^{-1}1{\cdot}055=46^\circ\ 32'$$

and Sec. β = Sec 46° 32' = 1·45

$$\therefore y=\frac{1}{635}\left(6x^2+\frac{35\times x^4}{2{,}500}\right)$$

From this equation, for

x=10′	20′	30′	40′	45′	50′
y=945′	3·81′	8·68′	15·68′	20·02′	25′

Stresses on Arch Ribs.

The additional dead load at the springing must now be corrected to agree with the more correct value of Sec β *viz*, 1·45

It then

=520+850 approx.
=1,370 lbs/lineal ft.

N B—This will produce the same result for tan β when adopted in the calculations for the arch axis curve, as small places of decimals only are affected therein

The stresses on a rib will now be calculated, using equations 10—26 (Chapter V, para 32 and Plate XXVII)

In these equations (see Chapter V, para. 30)

w_c=3,220 lbs/lineal ft

w_s=3,220+1,370=4,590 lbs/lineal ft.

$$\mu=\frac{w_s}{w_c}=\frac{4{,}590}{3{,}220}=1{\cdot}42$$

w=1,260 lbs./lineal ft.

Sec. β=1·45   r=25′   L=100′

h_c=22″=1·83′ h_s=36″=3′

b=40″=3·33′

$t^\circ=\pm30^\circ$ F.

Percentage of steel reinforcement in each case =0·5 per cent.

The results, calculated by slide rule, are tabulated below :—

FOR	Horizontal Thrust H = lbs.	Moment at Crown M_c = ft. lbs.	Thrust at Springing T_s = lbs.	Moment at Springing M_s = ft. lbs.	Shear at Springing V_s = lbs.
(i) Dead Load	1,72,300	±7,900	2,50,000	±18,800	1,83,500
(ii) Live Load for ±M_c	31,500	±1,15,000	..	..	..
(iii) Live Load for +M_s	50,500	..	73,000	+3,15,000	25,200
(iv) Live Load for —M_s	16,800	..	24,300	—2,52,000	42,000
(v) Temperature changes (using h_c).	±4,450	∓27,750	±3,070	±83,300	..
(vi) Arch shortening (H_s=1,72,300+$\frac{63,000}{2}$ +4,450=2,08,250).	—1,790	+11,200	—1,230	—33,700	..
(vii) Live Load over whole span.	63,000	..	..	..	63,000

The crown and springing sections must be designed to carry the worst probable combination of stresses, the resultant compressive unit stress f_c not to exceed approx. 600 lbs/sq. in. for net load results (i), (ii), (iii) and (iv), and f_c not to exceed approx. 750 lbs/sq. in. when allowing for temperature and arch shortening alterations (v) and (vi).

For the Crown Section :—

Take the combinations of corresponding values giving the greatest stresses from the tabulated results above. Design of Crown Section.

For dead load+live load excentricity of thrust from arch axis.

$$e=\frac{\text{Moment}}{\text{Thrust}}=\frac{M_c}{H}=\frac{7,900+1,15,000}{1,72,300+31,500}$$

$$=\frac{1,22,900}{2,03,800}=\cdot603'$$

$$\therefore \frac{e}{h_c}=\frac{\cdot603}{1\cdot83}=\cdot33$$

From Plate XXXIV, for ·05 per cent. steel in each face and $\frac{e}{h}=\cdot33,$

$$\frac{M}{b\,h^2 f_c}=\cdot127$$

$\therefore$ compressive unit stress $f_c=\dfrac{1,22,900\times12}{40\times22^2\times\cdot127}=600$ lbs./sq. in.

Allowing for temperature changes and arch shortening (the positive values for changes in M_c, combined with the correspond-

ing negative values for changes in H, obviously give the worst results)

$$e=\frac{M_c}{H}=\frac{1,22,900+(27.750+11,200)}{2,03,800-(4,450+1,790)}$$

$$=\frac{1,61,850}{1,97,560}=\cdot82'$$

$$\therefore \frac{e}{h_c}=\frac{\cdot82}{1\cdot83}=\cdot448.$$

Hence, from Plate XXXIV,

$$\frac{M}{b.h^2\,f_c}=\cdot131$$

$$\therefore f_c=\frac{1,61,850\times12}{40\times22^2\times\cdot131}=765 \text{ lbs./sq. in.}$$

which is near enough to the safe stress of 750 lbs./sq. in.

Design of Springing Section.

For the Springing Section:—

Take the combinations of corresponding values giving the worst stresses from the tabulated results above.

For dead load *plus* live load.

$$e=\frac{M_s}{T_s}=\frac{18,800+3,15,000}{2,50,000+73,000}$$

$$=\frac{3,33,800}{3,23,000}=1\cdot03'.$$

$$\therefore \frac{e}{h_s}=\frac{1\cdot03}{3}=\cdot343$$

Hence from Plate XXXIV,

$$\frac{M}{b\ h^2\,f_c}=\cdot128$$

$$\therefore f_c=\frac{33,800\times12}{40\times36^2\times\cdot128}=603 \text{ lbs./sq. in.}$$

$$=600 \text{ approx.}$$

Allowing for temperature changes, to give the worst results,

$$e=\frac{M_s}{T_s}=\frac{3,33,800+83,300}{3,23,000+3,070}=\frac{4,17,100}{3,26,070}=1\cdot28'$$

$$\therefore \frac{e}{h_s}=\frac{1\cdot28}{3}=\cdot43$$

Hence from Plate XXXIV,

$$\frac{M}{b\ h^2\,f_c}=\cdot13$$

$$\therefore f_c=\frac{4,17,100\times12}{40\times36^2\times\cdot13}=742 \text{ lbs./sq. in.}$$

$$=750 \text{ approx.}$$

Dimensions of Arch Rib.

As the stresses at both crown and springing are satisfactory, the rib section assumed, *viz.*, 22″ deep×40″ wide at the crown

and 36″ deep×40″ wide at the springing, will be adopted. (It is to be noted that several trials had to be made before this section was arrived at).

Vide plate XXVII, the depths of the arch rib at quarter points along its axis from the crown will be :—

$$\cdot 89h_c + \cdot 11h_s = (\cdot 89\times 22)+(\cdot 11\times 36) = 23\cdot 56''$$
$$\cdot 78h_c + \cdot 22h_s = (\cdot 78\times 22)+(\cdot 22\times 36) = 25\cdot 1''$$
$$\cdot 60h_c + \cdot 40h_s = (\cdot 60\times 22)+(\cdot 40\times 36) = 27\cdot 6''$$

Area of steel reinforcement at crown — Steel Reinforcement.

$$A_s = pbd = \cdot 005\times 40\times 22$$
$$= 4\cdot 4 \text{ sq. in.}$$

Vide Table XXXVI, use 6 bars of 1″ dia. in each face.

Area of steel reinforcement at springing

$$A_s = pbd = \cdot 005\times 40\times 36$$
$$= 7\cdot 2 \text{ sq. in.}$$

Use 9 bars of 1″ dia. in each face, *i.e.*, strengthen the reinforcement at the springing by 3 additional 1″ bars in each face.

These 3 bars will be stopped at $\frac{\text{span}}{10}$ =10 ft. above the springing.

All of the arch reinforcement bars must be carried down well into the abutments and piers, in order to develop the moment occurring at the springing. In this example (see Plate XXVIII) the reinforcement is continued to the footings of the piers. This is not essential, but it is desirable as strengthening the piers at small additional expense.

The main bars will be 3″ from the surface. No additional concrete is necessary to cover the bars, as is required in beams.

The arch rib main bars will be bound at 18″ centres by ½ diam. wire hooping.

All splices must be staggered and the bars lapped and wired for 50 diameters ; the ends of the bars must be hooked.

Full details of the design are shown on Plate XXVIII. — Details of Design.

See Chapter VI, para. 31, and Plates XXVII and XXVIII. — Setting out the Arch.

The arch axis is set out by plotting the values of x and y found above. Additional co-ordinates can be calculated and inter polated. In this case a 3 centered curve will be found to fit well over the points so plotted The change in the radius is at 20 ft. from the springing or 30 ft. from the crown.

From Plate XXVII and Chapter VI, para. 31.

$$\text{The centre radius } R = \frac{(ad)^2+(bd)^2}{2\ bd} \quad . \text{ (Equation 4).}$$

$$= \frac{30^2+8\cdot 68^2}{2\times 8\cdot 68} = 56\cdot 2 \text{ ft}$$

$$\sin \theta = \frac{ad}{R} \quad . \quad . \quad . \quad . \quad . \quad \text{(Equation 6)}$$

$$= \frac{30}{56 \cdot 2} = \cdot 534 = \sin 32^\circ\, 17'.$$

$$\therefore \theta = 32^\circ 17'$$

$$\cos \theta = \cdot 845$$

$$\text{The outer radius } R_1 = \frac{(ef)^2 + (ed)^2}{2\,(ed.\cos\theta - ef.\sin\theta)} \text{ (Equation 5).}$$

$$= \frac{20^2 + 16 \cdot 32^2}{2[(16.32 \times \cdot 845) - (20 \times \cdot 534)]} = 107 \cdot 1 \text{ ft.}$$

The diagram on Plate XXVII illustrates how the positions of the centres are found. The arch axis having been set out, the arch rib thicknesses or depths, as calculated above, will be laid off at 4 equidistant points along the arch axis. The centres for the intrados and extrados curves, which will be equidistant on each side of the arch axis centres, will be found by determining their centre and outer radii by trial and error. In this case, it will be found that these centres will come about $2\frac{1}{4}$ ft. and 8 ft. on each side of the arch axis centre.

Arch Rib Braces.

Arch Rib Braces.

The arch ribs are columns, and require braces between them at intervals not exceeding 8 times the width of the ribs, *i.e.*, in this case at intervals not exceeding $8 \times 40'' = 26'8''$.

The width of these braces parallel to the arch axes should be approximately the same as the mean depth of the arch ribs, *i.e.*, in this case 29″; their depth should be not less than $\frac{1}{10}$ of the distance between the arch ribs, *i.e.*, in this case not less than $\frac{1}{10} \times 13' = 1'\ 3''$ approximate.

The actual spacing of the braces must conform to the design of the bridge. Braces opposite alternate spandrel columns, *i.e.*, at intervals of 21′, will be suitable, of dimensions 2′ 6″ wide × 1′ 3″ deep, and reinforced with 0·5 per cent. steel in each face. 8 bars of $\frac{3}{4}''$ diameter will be suitable for the reinforcement; these bars will be carried across both arch ribs. Braces are not required at or near the crown, as the floor beams and flooring here brace the ribs sufficiently. There will therefore be four braces per span two on each side of the crown (see Plate XXVIII).

Wind Pressure.

Wind Pressure.

The arch need not be investigated for wind pressure, as the vertical surface is so small and the dead load proportionately so great, that wind pressure will cause but negligible alterations in the stresses.

PIERS AND ABUTMENTS.

The top widths of the piers will be made $=2h_s=6$ft., with a side batter of 18 over 1 to the foundation. Design of Piers.

In this case, the horizontal thrusts on the pier due to dead load, temperature changes, and arch shortening, balance.

The design will therefore be tested for dead load and the most unfavourable case of live load. The latter occurs under the conditions catered for in item (iii) of the tabulated stress results, in which the positive moment at the springing M_s gives a horizontal thrust H of 50,500 lbs.; this acts at a distance.

$\frac{M_s}{H}=\frac{3,15,000}{50,500}=6{\cdot}25$ ft. approx. above the centre of the springing.

The total vertical load on the pier, from one arch rib in each span.

=Dead load from two spans.
+Live load fron one span
+Weight of spandrel column above pier
+Weight of portion over the pier between the two arch axes.

$=2V_s$ (item (i) of tabulated stress results).

$+V_s$ for $+ M_s$ (item (iii) of tabulated stress results)

$$+22' \times \frac{24''+14''}{2} \times \frac{40''+14''}{2} \times 150 \text{ lbs. approx.}$$

$$\frac{2\frac{1}{2}+7}{2} \times 3\tfrac{1}{2} \times 3\tfrac{1}{2} \times 150 \text{ lbs. approx.}$$

$=(2\times 1,83,500)+25,200+12,000+8,700$

$=4,13,000$ lbs. approx.

Excentricity of resultant force on pier from centre of pier

$$=\frac{\text{Live Load Moment } M_s}{\text{Vertical load}}$$

$$=\frac{3,15,000 \text{ ft. lbs.}}{4,13,000 \text{ lbs.}}={\cdot}76 \text{ ft.}$$

As the pier is 6 ft. wide, the resultant force acts within the middle third, which is satisfactory.

Consider the unit stress on a section of the pier 6′ wide$\times$12 long.

$$\text{Direct stress}=\frac{\text{Load}}{b\times d}$$

$$=\frac{4,13,000}{12\times 6}=5,700 \text{ lbs./sq. ft. approx.}$$

Difference in stress on account of excentricity

$$=\frac{6\,M_s}{b\times d^2}$$

$$=\frac{6\times 3{,}15{,}000}{12\times 6}=44{,}000$$

$\therefore$ Maximum stress $=5{,}700+4{,}400$
$=10{,}100$ lbs./sq. ft.
$=70$ lbs./sq. in.

which is safe.

Consider the pier foundation, which will be approximately 25 ft. below the intersection of the arch axes.

Moment $=H\times(6{\cdot}25+25)$
$=50{,}500\times 31{\cdot}25=15{,}80{,}000$ ft. lbs. approx.

Total vertical load (from one rib in each span)
$=$ vertical load on $\frac{1}{2}$ pier
$+$ weight of $\frac{1}{2}$ the pier
$=4{,}13{,}000+$ say $(25\times 10\frac{1}{2}\times 29\times 150\times\frac{1}{2})$
$=4{,}13{,}000+5{,}71{,}000=9{,}84{,}000$ lbs.

Excentricity of resultant force from centre of foundation

$$=\frac{\text{Moment}}{\text{Vertical load}}$$

$$=\frac{15{,}80{,}000}{9{,}84{,}000}=1{\cdot}6 \text{ ft.}$$

As the foundation will be approximately 15 ft. wide, the resultant force cuts the foundation within the middle third, which is satisfactory.

Consider the unit stress on half the foundations, say 15′ wide $\times$ 17′ long.

$$\text{Direct stress}=\frac{\text{Load}}{b\times d}$$

$$=\frac{9{,}84{,}000}{17\times 15}=3{,}900 \text{ lbs./sq. ft. approx.}$$

Additional stress due to excentricity

$$=\frac{6\times M}{bd^2}$$

$$=\frac{6\times 15{,}80{,}000}{17\times 15^2}=25{,}000$$

$\therefore$ Approximate maximum stress
$=3{,}900+2{,}500$
$=6{,}400$ lbs./sq. ft.
$=2{\cdot}86$ tons/sq. ft.

This is a safe stress for any material on which an arched bridge foundation is permissible (See Chapter VI, para. 23).

It will be seen that a large difference in the value of the moment assumed above would not appreciably affect the design.

The above approximate method for arches does not introduce any large error, and such errors as occur are comparatively small and on the safe side.

The piers should also be analysed for live load on both adjacent spans, but such loading is not so severe in this case as the loading adopted in the above calculations.

The design of the abutments should be analysed similarly, including consideration of the increases in stresses due to temperature changes, and of the weight of miscellaneous bridge details and the effect of backfill pressure. Where the back face of the abutment is built against rock, the resultant force on the abutment must act within the middle third of the base plane at right angles thereto; otherwise the resultant must act within the middle third of the horizontal base plane. **Abutments.**

(See Chapter VI—design of piers and abutments, and Chapter IV—retaining walls).

Full details of the design are shown on Plate XXVIII. **Details of Design.**

In cases where the piers and abutments are founded on wells, they should be designed above the wells on the above principles. **Well Foundations.**

Example 10.

100 foot span reinforced concrete filled spandrel arch bridge.

See Plates XXVII and XXIX, and Chapter VI. paras. 22-37 also Volume I, Chapter VII, and Example 9.

Roadway 18′ wide, metalled.
P. C. concrete 1-2-4.
All steel to British Standard Specifications.
Live Load :—
For Floor—12 ton road roller+25 per cent. impact.
For Arch Ring—140 lbs. per sq. ft. of roadway.
Rise of Arch=15 ft.
Width of Arch=20 ft. approx.

Details for the spandrel walls and the arch ring will be calculated: the remainder of the design will conform to that worked out in Example 9, with suitable modifications: differences and details not referred to are illustrated in Plate XXIX.

Spandrel Walls.

(See Chapter IV.)

The spandrel walls are cantilevers resisting the lateral earth pressure of the filling with the superimposed load. **Spandrel Walls.**

Average angle of repose of rammed filling = 37°.

For this, from Table VII, ratio of horizontal to vertical pressure.

$C = \cdot 25$

The weight of the filling may be taken as 120 lbs./c.ft.

$\therefore$ equivalent lateral pressure

$w = \cdot 25 \times 120 = 30$ lbs./c.ft.

To allow for the concentrated load of a road roller, the wall will be designed for 3′ greater height above the road surface.

The depth of the fill varies from 17′ at the springing to 2′ at the crown.

$\therefore$ The height of the spandrel walls for design will vary from 20′ to 5′.

Design of 15′ height of spandrel wall.

For a 15′ height—1′ wide

$$\text{Moment} = \frac{WH^3}{6}$$

$$= \frac{30 \times 15^3}{6} = 16{,}800 \text{ ft. lbs.}$$

$$d = \sqrt{\frac{M}{95 \times b}} = \sqrt{\frac{16{,}800 \times 12}{95 \times 12}} = 13 \cdot 3''$$

Adding $1\frac{3}{4}''$,

Thickness $h = 15''$ at arch ring:

h tapers to 6″ at the top.

Area of reinforcement

$A_s = pbd = \cdot 0067 \times 12 \times 13 \cdot 3$

$= 1 \cdot 07$ sq. in.

From Table XXXVII, $\frac{7}{8}''$ bars at 7″ centres will do. Half of these reinforcement bars can be stopped about half way up.

Design of other heights of spandrel wall.

The remaining heights of spandrel wall will be designed similarly.

According as the wall is greater or less in height, the thickness h will be greater or less and the spacing of the bars will be less or greater.

ARCH RING.

Arch Ring Trail Section.

Try depth at crown $h_c = 20'' = 1' \cdot 67$

Try depth springing $h_s = 27'' = 2' \cdot 25$

Loads.

The loads and stresses will be worked out for a 12″ width of arch ring, the weight of kerbs, parapets and spandrel walls being taken as uniformly distributed over the full 20′ width of arch ring.

Dead load per lineal ft. at crown

$W_c = 505$ lbs./sq. ft.

Dead load per lineal ft. at springing

$W_s = 2{,}445$ lbs./sq. ft.

Live load $W = 140$ lbs./lineal ft.

Curve of arch axis

$$u = \frac{W_s}{W_c} = 4{\cdot}85$$

Inclination of springing line to horizontal

$$\beta = \tan^{-1}{\cdot}835 = 39^\circ\ 51'$$

Sec. $\beta = 1{\cdot}30$

Coordinates of arch axis curve

For

$x = 10'$	$20'$	$30'$	$40'$	$45'$	$50'$
$y = {\cdot}375'$	$1{\cdot}612'$	$4{\cdot}05'$	$8{\cdot}25'$	$11{\cdot}25'$	$15'$

Allow for changes in temperature $t = \pm 20^\circ$

Approximate stress results on a 12″ width of the arch ring.

Stresses on Arch Ring.

For	Horizontal Thrust $H =$ lbs	Moment at Crown $M_c =$ ft lbs.	Thrust at Springing $T_s =$ lbs	Moment at Springing $M_s =$ ft lbs	Shear at Springing $V_s =$ lbs
(i) Dead Load	69,000	±2,880	89,700	±5,050	57,600
(ii) Live Load for $+M_c$	5,850	±12,700	..	..	..
(iii) Live Load for $+M_s$	9,320	..	12,100	+46,700	..
(iv) Live Load for $-M_s$	3,110	..	4,050	—28,000	..
(v) Temperature changes.	±1,860	∓ 7,000	±1,430	±21,000	..
(vi) Arch Shortening ($H_a = 69{,}000 + \frac{11{,}700}{2} + 1{,}860 = 76{,}710$).	—1,260	+4,740	—970	—14,200	..
(vii) Live Load over whole span	11,700	..	..	.	7,000

$+M_c$ conditions will be taken for the design of the crown section as they are more severe than for $-M_c$ Design of Crown Section

For dead load+live load

$$e = \frac{M_c}{H} = \frac{2{,}880+12{,}700}{69{,}000+5{,}850} = \frac{15{,}580}{74{,}850} = {\cdot}208'$$

$$\therefore \frac{e}{h_c} = \frac{{\cdot}208}{1{\cdot}67} = {\cdot}125$$

From Plate XXXIV,

for 0·5 per cent. steel reinforcement and $\frac{e}{h} = {\cdot}125$

$$\frac{M}{b h^2 f_c} = {\cdot}085$$

$$\therefore f_c = \frac{15{,}580 \times 12}{12 \times 20^2 \times {\cdot}085} = 458 \text{ lbs./sq. in.}$$

This is well within the permissible limit of 600 lbs./sq. in.

Allowing for temperature changes and arch shortening,

$$e = \frac{M_c}{H} = \frac{15{,}580+(7{,}000+4{,}740)}{74{,}850-(1{,}860+1{,}260)}$$

$$=\frac{27,320}{71,730}=\cdot 38'$$

$$\therefore \frac{e}{h_c}=\frac{\cdot 38}{1\cdot 67}=\cdot 228$$

Hence from Plate XXXIV

$$\frac{M}{bh^2 f_c}=\cdot 118$$

$$\therefore f_c=\frac{27,320\times 12}{12\times 20^2\times\cdot 118}=580 \text{ lbs./sq. in.}$$

This is well within the permissible limit of 750 lbs./sq. in.

Owing to the margins in the resultant compressive unit stresses in the concrete as compared with the permissible safe unit stresses, a 20″ depth at the crown is unnecessary.

The depth of the arch ring at the crown may be reduced to approximate.

$$20\times\sqrt{\frac{458}{600}}=17\cdot 5''$$

$$\text{or } 20\times\sqrt{\frac{580}{750}}=17\cdot 6''$$

Make the crown depth $h_c=18''$.

Design of Springing Section.

$+M_s$ conditions will be taken for the design of the springing section, as they are more severe than for $-M_s$.

For dead load+live load.

$$e=\frac{M_s}{T_s}=\frac{5,050+46,700}{89,700+12,100}$$

$$=\frac{51,750}{101,800}=\cdot 51'$$

$$\therefore \frac{e}{h_s}=\frac{\cdot 51}{2\cdot 25}=\cdot 227$$

$$\text{Hence } \frac{M}{bh^2 f_c}=\cdot 118$$

$$\therefore f_c=\frac{51,750\times 12}{12\times 27^2\times\cdot 118}=600 \text{ lbs./sq. in.}$$

=safe limit.

Allowing for temperature changes to give the worst results

$$e=\frac{M_s}{T_s}=\frac{51,750+21,000}{1,01,800+1,430}$$

$$=\frac{72,750}{1,03,230}=\cdot 705'$$

$$\therefore \frac{e}{h_s}=\frac{\cdot 705}{2\cdot 25}=\cdot 313$$

Hence from Plate XXXIV

$$\frac{M}{bh^2 f_c}=\cdot 127$$

$$\therefore f_c = \frac{72{,}750 \times 12}{12 \times 27^2 \times \cdot 127} = 790 \text{ lbs./sq. in.}$$

which is above the safe limit of 750 lbs./sq. in.

Also the reinforcement steel tensile stress is less than 15 f_c *i.e.*, less than $15 \times 790 = 11{,}800$ lbs./sq. in., which is uneconomical. (See Volume I, Chapter VIII.)

To reduce the compressive unit stress in the concrete to the safe limit of 750 lbs./sq. in., the depth of the arch ring at the springing must be increased to approximately.

$$27 \times \sqrt{\frac{790}{750}} = 27 \cdot 9''$$

Make the springing depth $h_s = 28''$.

The arch ring will therefore be 18″ deep at the crown, and 28″ deep at the springing. Dimensions of Arch Ring.

Vide Plate XXVII, the depth of the arch ring at quarter points along its axis from the crown will be

$\cdot 89h_c + \cdot 11h_s = (\cdot 89 \times 18) + (\cdot 11 \times 28) = 19 \cdot 08''$

$\cdot 78h_c + \cdot 22h_s = (\cdot 78 \times 18) + (\cdot 22 \times 20) = 20 \cdot 15''$

$\cdot 60h_c + \cdot 40h_s = (\cdot 60 \times 18) + (\cdot 40 \times 28) = 22''$.

Area of steel reinforcement at crown Steel reinforcement.

$A_s = pbd = \cdot 005 \times 12 \times 18 = 1 \cdot 08$ sq. in.

From Table XXXVII, 1″ diameter bars at 9″ centres will be suitable, in each face

Area of steel reinforcement at springing

$A_s = pbd = \cdot 005 \times 12 \times 28$

$= 1 \cdot 68$ sq. in.

Deducting the area of the two 1″ bars, the area of the additional reinforcement necessary at the springing $= 1 \cdot 68 - 1 \cdot 05 = \cdot 63$ sq. in.

From Table XXXVII, use ¾″ bars at 9″ centres, in each face. These ¾″ bars will be stopped at $\frac{\text{span}}{10} = 10$ ft. above the springing.

All of the bars will be carried down into the abutments to a depth=80 diameters, *i.e.*, 6′8″ in the case of the 1″ bars and 5′ in the case of the ¾″ bars. The bars will be 2½″ from the surface.

⅜″ diameter binding bars, encircling the two layers of reinforcement bars, will be inserted at intervals of 2½′, to bind the material of the arch ring together.

Full details of the design are shown on Plate XXIX. Details of Design.

See Chapter VI, para. 31 and Plates XXVII and XXIX.

The arch axis is set out by plotting the values of x and y found above. Setting out the Arch.

In this case a 3 centred curve will be found to fit the points, departing from the exact curve not more than $\frac{1}{50}$ of the arch ring depth. The change in the radius is at 20 ft. from the springing or 30 ft. from the crown.

From Plate XXVII and Chapter VI, para. 31.

The centre radius

$$R = \frac{(ad)^2 + (bd)^2}{2\ bd}$$

$$= \frac{30^2 + 1{\cdot}05^2}{2 \times 4{\cdot}05} = 111{\cdot}3 \text{ ft}$$

$$\text{Sin}\ \theta = \frac{ad}{R}$$

$$= \frac{30}{111{\cdot}3} = {\cdot}2656 = \text{Sin}\ 15^\circ\ 21'$$

$$\therefore \theta = 15^\circ\ 21'$$

$$\text{Cos}\ \theta = {\cdot}9641$$

The outer radius

$$R_1 = \frac{(ef)^2 + (ed)^2}{2\ (ed\ \cos\theta - ef\ \sin\theta)}$$

$$= \frac{20^2 + 10{\cdot}95^2}{2\ [(10{\cdot}95 \times {\cdot}961) - (20 \times {\cdot}266)]} = 49{\cdot}6 \text{ ft}$$

The arch ring depths, as calculated above, will now be laid off at 4 equidistant points along the arch axis, and the centres for the intrados and extrados found by trial and error, and the intrados and extrados finally plotted, as in Example 9.

N. B.—In this Example R is greater than R_1 whereas in Example 9 R_1 is greater than R. This difference is due to the difference in the distribution of the dead load along the span in the cases of the filled and open spandrel designs.

CHAPTER VIII.

Causeways and training works.

Introductory—Causeways—Training and Protection Works.

INTRODUCTORY.

1. General instructions regarding the use and principles of construction of causeways and protection works are given in Chapter II.

Methods of calculation and design are given in this Chapter.

Substantial construction, intelligently designed with due regard to the conditions to be met, is essential in all cases.

CAUSEWAYS.

2. The main principles in the design of a causeway are that it must not contract the stream, that it must maintain a firm roadway against damage by floods, erosion, or movement of the *nala* bed, and that it should be set out at right angles to the current, to avoid scouring along the upstream wall. **Principles of construction of causeways.**

In wide straggling *nalas* with two or more streams, these should when practicable be trained through one causeway of length equal to the aggregate width of all the streams at observed high flood level. Small subsidiary streams can sometimes be provided for by a culvert in the embankment. The embankment if liable to erosion by floods must be protected by dry stone walling or pitching carried at least 3 ft. below the *nala* bed level and $1\frac{1}{2}$ ft. above observed high flood level.

Causeways should be set out on the site by an officer; this duty must not be left to a subordinate.

3. Causeways may be built of masonry, concrete, or reinforced concrete. The reinforced concrete type, though more troublesome to build, has the advantages that it is not liable to fail in detail, even after severe scour, and that its maintenance and repair costs will be small. **Types of causeways.**

Causeways may be designed with deep upstream and downstream walls with no protection work, or with shallow walls, protected by boulder filled wire mattresses, or by a reinforced concrete slab apron, below the downstream wall.

Protection work of any kind is costly, and requires repairs after every flood, and more excavation during construction. It is cheaper to construct the up and downstream walls deep enough to be beyond scouring action, and for this purpose sufficient

suitable timbering and powerful pumps for unwatering must be provided.

Details of causeways. 4. Constructional details for causeways are given in Plate XXXIX, Figures 1 and 2.

These designs illustrate 18′ wide causeways (suitable for Class I and II roads) constructed in masonry with cement concrete flooring, or in reinforced concrete throughout, and the formulæ are also given for the calculation of the depths of the drop walls, in terms of the average depth of water over the causeway at high flood.

Upstream Wall. 5. The upstream wall foundations must be taken below scour level, which may be ascertained by trial pits.

A rough rule is that the depth of scour below bed level = half the depth of water at observed high flood $= 1\frac{1}{2}$ d.

The foundations should be taken $1\frac{1}{2}$ ft. below the scour at the deepest point, subject to a minimum total depth of $3\frac{1}{2}$ ft. Where the upstream wall is not normal to the current, the foundations should be 50 per cent. deeper.

The thickness of the masonry should be one-third of the height of the wall, *i.e.*, $= \frac{h}{3}$

Downstream Wall. 6. The downstream wall is subject to the scouring action of the water, which forms a water cushion.

A water cushion formula for falls in canals with small velocities is

$$x = h + 1\cdot5 \sqrt[3]{d} \sqrt{h}$$

where x = depth of cushion below upper floor of fall or causeway.

h = fall in water surface at the waterfall or causeway.

v = velocity of stream in ft. per second.

d = average depth of water.

In swiftly flowing streams, however, this formula must be modified to allow for the velocity.

Hence for causeway

$$h = \text{velocity head} = \frac{V^2}{64\cdot4}$$

and instead of d use d h in the above formula

$$\therefore x = h + 1\cdot5 \sqrt[3]{d+h} \sqrt{h}$$

For rapid streams $h = \frac{d}{2}$ approximately and $x = 1\frac{1}{2}$ d.

Foundations should be taken to $1\frac{1}{2}$ d + 4 ft., and not less than 2d below the edge of the causeway in bajri beds. In sandy beds these depths may require to be increased.

The depths of water cushions formed on existing causeways during floods should be observed and recorded for guidance.

The thickness of the masonry should be one-third of the height of the wall, *i.e.*, $=\frac{H}{3}$.

7. Where, after practical experience of floods, protection mattresses prove necessary, mattresses in wire trangaring should be securely attached to the eyebolts already embedded in the downstream wall (see Figure 1 on Plate XXXIX), extending downstream to a distance=H (*i.e.*, not less than 2d), or the observed length of scour if greater. **Protection Mattresses.**

8. The surface of the causeway should follow the nala bed level and the slope of the banks, all changes in gradient being eased off by vertical curves. **Surface of causeways.**

The causeway cross surface should slope downstream at the same slope as the nala bed, subject to a maximum of 1 in 30.

The upstream and downstream walls should be rounded on their outer edges, and they should enclose and be flush with the causeway surface.

9. The slopes or ramps at the ends of the causeway should follow the bank, with a maximum gradient of 1 in 14, and be continued to 1½ ft. above observed high flood level. Where the banks are steeper than 1 in 14, no part of the ramp below observed high flood level should project beyond the bank. **Ramps.**

Where the ramps of existing causeways project into the stream, pyramidal boulder filling should be placed across the angles between the upstream sides of the ramps and the banks.

10. These are put in at changes of gradient according to the method shown on Plate XXXIX, Figure 3, by lowering the road or causeway surface at salients, and raising it at re-entrants. **Vertical Curves.**

An example is as follows :—

Approach at 1 in 110 down

End of causeway at 1 in 14 down, length 80 ft.

Causeway from foot of 1 in 14 slope is at 1 in 46 down.

Then for upper curve $x=\frac{1}{n}-\frac{1}{m}=\frac{1}{14}-\frac{1}{46}=\cdot 05$

$$\text{and } D=\frac{3}{x}=\frac{3}{\cdot 05}=60 \text{ ft.}$$

i.e., if the 1 in 14 slope were 48+60=108 ft. long or longer, these two curves could be put in and the road lowered and raised on the upper and lower curves respectively by the amounts shown. If the slope were longer than 108 ft., there would be a straight part at 1 in 14 between the ends of the curves. But as the total length is 80 ft., D and the amounts by which the road is lowered or

raised must be all multiplied by $\frac{80}{108}$ = ·74

and ·74 D (48 and 60 ft.)=36 ft. and 44 ft. so that the road must be lowered or raised by the amounts shown on Plate XXXIX multiplied by ·74, *i.e.*, by

·037′ ·14′ ·31′ ·55′.

The curves can be lengthened, and the alterations to the road surface increased, in a similar manner.

The same method is used for easing off changes of gradient in the causeway proper.

Scuppers. 11. The uses of scuppers, which are a miniature form of causeway, are given in the general specification (Chapter II).

Scuppers correspond to causeways as culverts to bridges. They extend across the entire formation width.

They may be surfaced with dry stone, stone in lime concrete, or cement concrete on lime concrete. Dry stone surfacing should not as a rule be used except upon unmetalled cart roads and pack transport roads. Cement concrete surfacing is usually the best for scuppers on M. T. roads (they should not be used in Class I roads), as in the case of causeways, where not unduly expensive.

A scupper should be laid down in three curves in the direction of the road alignment, convex at the ends and concave in the middle, with the requisite cross slope of 1 in 12 to 1 in 30. Longitudinal and transverse hill road sections, illustrating scuppers, are given in Plate XL.

Specifications. 12. The materials and workmanship in all road structures (including causeways and scuppers) must conform to the specifications laid down for building work generally (see Volume I Buildings and General), as applicable, amplified by the specifications given in this Chapter and in Chapter VI. All road structure work demands the best materials and workmanship, in an especial degree.

TRAINING AND PROTECTION WORKS.

Classes of training works. 13. Training and protection works are provided :—

(i) To protect bridge or causeway approaches from erosion, and to guide the current straight through or across the bridge or causeway.

(ii) To protect an exposed road embankment, running along the bank of a stream or otherwise liable to erosion by floods.

General principles relating to training and protection works are given in the General Specifications (Chapter II).

See River Training and Control on the Guide Bank System by F. E. Spring.

14. Training bunds will usually be necessary for bridges and causeways where the waterway is contracted by the abutments or by the banks, or where tributaries or bends or other causes above the site produce diagonal currents. **Bridge and causeway training works.**

The Bell Bund system is generally suitable.

Bridge or causeway training bunds should be parallel to the main current for a length upstream equal to the length of the bridge and downstream for $\frac{1}{10}$ to $\frac{1}{5}$ of this length.

The exposed ends of training bunds should be curved back at 140° to 120°, and be provided with impregnable heads of ample dimensions to safely resist all floods.

15. Road embankments liable to longitudinal scour and erosion are best protected by bunds built out into the river bed at a slight angle, not exceeding 30°, with the current behind and below which silting will occur. **Embankment protection works.**

The heads of these bunds must be well built into the bank. They should not project too far into the river bed, and a series of such bunds is often necessary, the intervals being determined by their angle and length, and consequent protective capacity, having regard to the silting effect produced. The toes or lower ends of these bunds must be made as impregnable as possible against scour.

Intervals, angles, and lengths can only be determined by study and experience, with particular regard to the type of stream or flood, the nature of the bed, and local conditions generally in each case.

As a rough exemplary rule, where the main current is approximately parallel to the bank, bunds 100 ft. long at an angle of 30° with the bank or current may be sited 150 feet apart, in a silting river or nala of average velocity.

Short stout projecting spurs or groins are sometimes used for this purpose, to break up the current and stop erosion by causing silting between the spurs. As these obstruct and force the water rather than lead it, their preferential use is not advocated except in special cases. For slow moving overflow floods in flat country, pitching, combined where necessary with spur bunds, is generally suitable (see also Chapter X, para. 31).

Where, as sometimes occurs in hill roads, a section of the road has to be built in embankment along a torrential or fast river bank, at a point where the full force of the current impinges, (*e.g.*, on the outer side of a general curve in a contracted river bed alignment), a pucca stone masonry retaining wall is necessary, the foundations being taken down to twice the depth of maximum scour and protected where necessary by an apron of boulder mattressing.

16. A type of training bund is illustrated in Plate XLI. **Design of Bunds.**

The slopes should not be steeper than from 1/1 to 1/1½. The stone pitching on the exposed face must be hand packed, and unless boulders large enough to resist movement by the current (see Chapter V) can be definitely procured in sufficiently large quantities, the pitching should consist of the largest stones available packed in wire mattressing or trangaring of suitable thickness (1-2 feet); individual mattresses should not be less than 6′×3′ in size, and must be securely joined together.

The protective pitching must be carried round the exposed end or ends of a bund.

A boulder mattress apron should be provided, for protection against scour, along the exposed toe of the bund in cross section, well fastened into the interior of the bund.

As a general rule, the width of the apron should be 1½ times the scour, increased at exposed ends to twice the scour, and where exceptional velocities are to be coped with to 2½ times the scour.

The thickness of the apron should equal the thickness of the pitching (1 to 2 feet). It is sometimes advisable to increase this thickness 50 per cent. at the outer edge, in the form of a square.

Wire Crate Work.

17. The following specification describes the construction of wire crates. This is applicable, with suitable modifications, to boulder mattressing for the protection of the toes of training bunds or footings of retaining walls impinged upon by a stream as well as to self-contained boulder crate protection works, which are particularly used to protect existing structures.

The usual unit size for a boulder crate, considered *per se*, is 10′×5′×4′; small crates are preferable where there is danger of unequal subsidence or overturning, or where they have to be dropped into deep water, and in such cases the usual 10′×5′×4′ crate is divided into two 5′ compartments by cross netting. A convenient size for mattresses, which, as already remarked, should not be of smaller unit size than 6′×3′×1′ thick, is 25′×10′×2′, but the dimensions depend upon the individual requirements; mattresses in excess of the latter size are inconvenient to handle and to construct of adequate strength. Large mattresses of this nature should be securely tied together at 5′ intervals to prevent bulging, in any case.

The cage or trangaring should ordinarily be made of No. 6 or No. 8 S. W. G. galvanized iron wire (smaller sizes of wire are too thin for use with boulders in hill torrents) woven with a 6″ to 4″ hexagonal mesh. Wherever possible they should be placed in position before being filled with boulders; after filling the top is securely wired all round to the sides. In self-contained crate bunds the boulders used for filling should average a foot in diameter, if possible, (in other cases the largest boulders available must be used, with 1′ boulders on the outside if possible—never so small

as 6″, or the meshing must be correspondingly smaller). The boulders must be carefully hand-packed, as tightly as possible; they must not be merely thrown in.

The weaving is done as follows for a 6″ mesh :—

(See Plate XL.)

A row of spikes is driven at 6″ intervals into a beam rather longer than the width of the netting. The wire is then cut into lengths about three times the length of the netting required, and each piece is bent in the middle round one of the spikes so as to form two stands which are straightened out on the ground. The weaving is then started from one corner — a *double* twist being given at each intersection as shown in the diagram. The bottom and two ends of a crate are woven in one piece and the ends bent round, after which the sides and any intermediate partitions are fixed in position, and secured by twisting adjacent wires round one another—at least two complete turns being given. This is done with an iron bar, as shown in the diagram, and is the part of the work requiring most careful supervision, as, if these junctions are not properly secured, the crates will simply open out once there is any movement. Approximately 65 lbs. of wire are required per hundred square feet of netting of 6″ mesh. A finer mesh must be used when only small material is available for filling the crates.

Details of a method of wire mattress construction adopted with success in the N. W. Frontier Province are given in Appendix LVII and illustrated in Plates LVIII—LX.

Old bhoosa bale hoop iron is often used with success for caging boulder bunds or mattresses.

Wire netting is unduly expensive, and, alternatively, weak.

For temporary or emergent protection work, country rope trangaring or caging may be used; this of course cannot be depended upon to last, but it sometimes fulfils its purpose when the resultant silting obviates the necessity for its replacement by a more permanent form of protection work.

18. Where scour threatens to undermine the foundations of piers or abutments, they may be protected by a flexible apron. This may consist of boulders in wire mattressing, as described under training bunds, or of cement concrete blocks, which are suitable except in the case of river beds composed of boulders without a fair admixture of sand or gravel. Where concrete blocks are used, a suitable type consists of a series of rings of such blocks, the blocks being of dimensions about 4 ft.×4 ft.×6 inches thick, and connected in both directions by steel cables. These aprons should be fixed at a point not less than 3 feet above foundation level, and be wide enough overall to afford a slope of 1 in 3

Protection of existing structures.

down to maximum scour level. The radial cables must be well anchored into the pier or abutment.

(See also para. 17).

19. An alternative method is to build a wall of piles across the river immediately below the bridge, and thus encourage silting around the piers and abutments. Such a wall must extend fully across the river, and its summit should be brought to average bed level. As the scour on the downstream face of the wall will be considerable (as in a causeway), the wall foundations must extend below scour level, and the wall must be substantial enough to withstand the water pressure.

20. In certain cases a series of obstructions placed in the path of the main current above the bridge may be so arranged as to spread the current at the bridge.

21. A ring of piling driven completely around the threatened pier or abutment may also be effective, where the bed is suitable for piles. (See Chapter VI.)

CHAPTER IX.

Road signs.

Road Direction Posts—Warning Signs and Notices—Village, Place and Road Name Signs—Mile and Furlong Stones.

1. Instructions regarding the provision of Road Signs on roads of various classes are given in Chapters II and III.

2. Road signs are of the following descriptions :— Classes of Signs.

Road Direction Posts.
Warning Signs and Notices.
Village and Place Name Signs.
Road Name Signs.
Mile Stones.
Furlough Stones.

The standard patterns of signs are shown in Plates XLII, XLIII, XLIV and XLV.

3. The careful selection of the most suitable position for the direction post is of great importance, so as to secure the maximum visibility on all the roads of approach. In certain cases it may be advisable to resite existing posts to secure a more dominant position. Road Direction Posts.

It is generally undesirable to mask the lower portion of the post in hedges or shrubs. The full length of the post should be visible wherever possible.

The projection of the direction arms over the roadway should be avoided.

The direction arms should be set at such angles on the head of the post as to ensure that each arm shall lie along the immediate general direction of the road it is indicating.

In all cases the higher arms should indicate the more important roads and only the arms indicating the same road should be set in the same horizontal plane.

If wooden or metal posts, other than posts of some standard steel section, are used, the top and bottom dimensions of the post as figured in the diagram should be taken as a guide.

The length of the arm for new direction posts will be mainly dependent upon the number of letters in the longest place name. Due regard should be had to the spacing between words and the proportions of the letters as figured on the diagrams.

The following standard dimensions and details should be followed :—

(*a*) Height of arm from ground :—Minimum 8′, maximum 9′ 9″.

(*b*) Length of arm (variable) :—Minimum 3′.

(*c*) Depth of arm :—Minimum 7″.

(*d*) Separation between arms :—Minimum 3″.

(*e*) Lettering :—Black block letters raised $\frac{1}{8}$″ on white ground.

For single line or upper of two lines :—3″ letters, $\frac{7}{16}$″ thick, $\frac{1}{2}$″ interval between letters.

For lower of two lines :—1$\frac{1}{2}$″ letters, $\frac{1}{4}$″ thick, $\frac{3}{16}$″ interval between letters, 1$\frac{1}{2}$″ space between lines.

(*f*) Post :—Painted plain white.

(*g*) An indication may be given on the post of the Road Authority responsible for its maintenance.

(*h*) In all but exceptional cases the arm should be lettered on both sides, the nearest important town being given first, then the nearest small town. The mileage should be given in figures only, immediately following the place to which it refers, the lowest fraction being a quarter.

(*i*) Wherever possible the direction post should be placed in such a position as to be visible to traffic from all converging roads for a distance of at least a hundred yards.

(*j*) The authority having control over the more important roads should be responsible for the provision and maintenance of the necessary direction posts at the road junctions.

Road direction posts are illustrated in Plate XLII.

Warning Signs and Notices.

4. A few signs, in carefully selected positions, and intended definitely to control cases where caution is imperative, will have a greater effect than the indiscriminate use of a large number of signs of varying shapes and colours.

Importance is attached to the careful selection of the site, and the placing of the post clear of bushes or other obstruction to visibility, so that the full length of the supporting post is seen. Where a choice of position exists due regard should be paid to the background for showing up the sign. For this reason, save where posts of some light standard steel section are used, the dimensions of the post as shown on the diagram are suitable.

The special danger to be guarded against should be indicated by means of a clear and legible symbol, based on the international symbols as far as applicable.

The list of dangers to be notified in this manner is as follows :—

(1) "Cross Roads."
(2) "Corner," right hand turn.
(3) "Corner," left hand turn.
(4) "Double Corner."
(5) "Level Crossing."
(6) "Road raised," *e.g.*, across a canal distributary.
(7) "River crossing" or "Irish bridge" (Causeway).
(8) "Road up" or "Bad road."
(9) "Steep hill."
(10) "Speed limit."
(11) "Hairpin bend" (on hill roads only).

The appropriate signs are illustrated in Plates XLIII—XLV.

The sign for "Double Corner" is reversible for a double corner in the opposite directions to those illustrated, *i.e.*, a left hand turn followed by a right hand turn. The sign for a steep bend—almost a hairpin bend—is also reversible to indicate a bend in the reverse direction to that shown. The illustrations for both signs show a right hand turn. The form of base for sign (8) is for use only when the sign is utilized as a "Road up" sign.

The symbol should be regarded as the principal means of indicating the nature of the danger to be guarded against.

Cast iron is recommended for the material of the plates, with letters and details in relief. In the event of the cost of this being prohibitive sheet iron or boards may be used.

In towns and suburban areas, where street lighting arrangements permit, the illumination of road signs is desirable. At certain special danger points upon roads of the above character it may be found desirable to erect special illuminated signs of the glass fronted, internally illuminated, type. In these cases, the red triangle and appropriate symbol, where used, should be enlarged to a uniform size of 1½ times the standard size.

The following standard dimensions and details should be followed :—

(*a*) Height to the underside of the triangle 7′ 9″. Length of sides of triangle 18″. Colour of triangle "post office" or "signal" red.

Symbols to be of standard pattern, and to be painted white.

(*b*) Space between triangle and top of caution plate 6″.

(*c*) Size of caution plate 18″×6″ overall, 2″ letters ¼″ thick with 1″ intervals between letters; border ½″ wide all round. Letters and border to be raised ⅛″ and painted red or black on white field. Height of lower edge of caution plate from ground 6′9″.

(*d*) Caution plates should not be exhibited without symbols

(e) All warning signs and notices to be placed facing approaching traffic and on near side of road and, wherever possible, to be set clear of obstructions so as to display the full length of the supporting post. They should be sited as closely as possible to a distance between 75 and 100 yards from the object of danger or commencement of the danger zone; and, if possible, be so placed that on either side of their positions a length of roadway and margin is clear and free from any obstruction to view such as lamp posts, telegraph poles, tramway standards, trees, etc.

(f) The supporting posts should be of iron or other suitable material painted white, and firmly embedded in the ground.

(g) "Cross Road" warning signs are not necessary where there are direction posts at cross roads and junctions in visible positions. The approaches to towns and villages should need no warning sign as such inhabited places are a sufficient indication in themselves that special care is necessary.

Village and place name signs.

5. These are erected on the main approaches to towns and villages, giving the name of the town or village. Standard types are illustrated in Plate XLII.

The dimensions of the supporting post at top and bottom shown in the diagram are optional; these dimensions should not be substantially less than the figures given, unless posts of some light steel standard section are used. The form and proportion of the letters will, of course, determine the length of the plate. Certain place names of more than average length, *e.g.*, two long words with a hyphen between, could be conveniently arranged in two lines.

The size of letter given in the diagram should always be regarded as a minimum.

The following standard dimensions and details should be followed:—

(a) Height to centre of name on plate:—Minimum 7′.

(b) Depth of plate:—9″.

(c) Height of letters 6″. To be raised $\frac{1}{8}$″ painted black on white field, down stroke $\frac{5}{8}$″ thick upstroke $\frac{5}{16}$″ thick; $\frac{1}{2}$″ space between letters; $1\frac{1}{2}$″ to 2″ clearance top and bottom.

(d) To be sited on near side of road facing approaching traffic, and at a distance of approximately 100 yards or thereabouts from the first houses of the village or town.

(e) The supporting post to be of iron or other suitable material, painted white, and firmly embedded in the ground.

6. A useful form of road name sign, carried out in reinforced concrete, is illustrated in Plate XLII. **Road name signs.**

The plate should be set at such an angle to the general direction of the road it is indicating as to be clearly visible from all approaches at a distance of about 50 yards.

Projection of the plate over the roadway or sidewalk is undesirable.

It is undesirable to mask the lower portion of the post, and the full length should be visible wherever possible.

The length of plate will depend upon the number of letters in the road name. Due regard should be had to the spacings between words and letters, also to the proportions of the letters.

The following standard dimensions and details should be followed :—

(*a*) Height of plate from ground :—7′ to centre of plate.

(*b*) Depth of plate :—9″.

(*c*) Lettering :—Black block lettering raised (or sunk) $\frac{1}{8}$″ on white ground. Letters 6″ high and $\frac{5}{8}$″ thick with $1\frac{1}{2}$″ space between letters, $3\frac{1}{4}$″ space between words, $1\frac{1}{2}$″ margin above and below letters.

(*d*) Post :—To be of iron or other suitable material, painted white.

(See also Chapter II, para. 38, and Chapter III, paras. 8 and 18).

7. Suitable standard types for mile stones are illustrated in Plates XLVI and XLVII. **Mile and Furlong stones.**

Furlong stones should as a rule be of hexagonal or square cross section, and of a size not to be confused with mile stones.

Mile and furlong stones should be whitewashed; the letters and figures on them should be countersunk, and painted black.

They may be made of cement concrete or quarried stone. The latter is often preferable, as concrete mile and furlong stones are susceptible to wilful petty damage to a greater extent than stone.

(See also Chapter II, para. 37, and Chapter III, para. 17.)

CHAPTER X.

Metalling and Road Maintenance.

General Remarks—Metalling—Scarifying Method of Remetalling—Rut Repair Method of Remetalling—Steam Rollers—Maintenance of Road Surface—Special Methods of Road Surfacing (Oiling, Tarring, and Concreting)—Maintenance of Berms—Maintenance of Drainage—Maintenance of Road Structures and Road Signs—Road Gangs—Maintenance of Boat Bridges—Traffic Control—Maintenance of Unmetalled Roads—Road Improvements,

General Remarks.

Introduction

1. These instructions should be read in conjunction with the instructions contained in the General Specification for the construction of roads given in Chapter II, and with the notes on repair estimates in Chapter XI. They are primarily of application to metalled roads, but should be followed, where applicable in the case of unmetalled roads, regarding which notes are given at the end of this Chapter.

Road Records.

2. A record plan, a metalling chart, and a road structure inspection book should be maintained for each road.

The record plan should be on a scale of 12" to 1 mile longitudinally, transverse measurements being exaggerated if necessary for clarity. This plan should show all important particulars of the road, including gradients, radius of curves, metal stacking places in hill sections, catchwater drains, mile and furlong stones, boundaries of land acquired for the road, scuppers, culverts, causeways, bridges, and buildings, cross roads, and all conspicuous objects along the road.

The standard pattern of metalling record chart, and also a useful pattern of current progress chart, are shown on Plate XLVIII.

The road structure inspection book should be maintained with records of all special repairs necessary and done to scuppers, culverts, causeways, bridges, mile stones, etc. All road structures should be legibly numbered and recorded by their numbers in the Inspection Book. They should be numbered in series by miles; thus 6/15 denotes the sixth culvert, etc., in the 15th mile.

In respect of every important river or nala crossing on every road, whether bridged or not, a cross section of the river or nala at the road crossing should be maintained, appended to the road structure inspection book (usual scale 1/20" to 1/40" horizontally and 1/10" to 1/20" vertically, see Chapter V, para. 10). On these

cross sections full particulars of heavy rainfalls and floods, with dates, are recorded as they occur. Rain gauges should be maintained at such places, and highest maximum flood levels, with dates, should be marked on bridge abutments and/or on the banks, as practicable.

3. *Scarifying* consists in digging up the old metalling surface to a depth of some 3 inches, preparatory to remetalling. It can usually only be done economically by means of a "scarifier" drawn by a suitable tractor (a steam roller will do). Definitions.

Scoring consists in roughening the old surface with pickaxes prior to spreading the new metal.

Special repairs are extensive repairs to any drain, parapet wall, bridge, culvert, etc., the cost of which exceeds Rs. 200 for the particular item concerned. They should be financed from reserves held locally by each Assistant Commanding Royal Engineer, and allowed for in the maintenance estimate, and in cases of special unforeseen damage due to cloud bursts, etc., from similar reserves held by superior officers.

4. In November and the early part of December of each year every Assistant Commanding Royal Engineer should make a point of traversing all his metalled roads with the Garrison Engineers or independent Sub-Divisional Officers concerned, and settling what work under the following heads should be included in the estimates for the following year :— Annual inspection of roads.

(*a*) Remetalling with one layer of 4½ inches spread.
(*b*) Scarifying and remetalling.
(*c*) Continuous rut repairs.
(*d*) Collection of metal only for either (*a*), (*b*), or (*c*).

It rests with him to consider all the roads in his charge as a whole, and to see that a suitable standard of excellence is maintained. If the decision on these points is left to subordinates without the Assistant Commanding Royal Engineer checking their demands at site, and at about the same time of year, certain roads will probably be starved at the expense of others.

When making this special inspection the Assistant Commanding Royal Engineer should also note (i) what "special" repairs will be necessary ; (ii) that stacks of patching metal are as laid down, (iii) that milestones are correctly marked with dates of remetalling ; (iv) that the remetalling chart is up-to-date ; (v) that the road structure inspection book has been kept up.

See also Chapter XI, para. 17, etc.

METALLING.

5. The normal period for remetalling, under normal conditions of traffic, is once every 4 years, on the average. The exact When remetalling is necessary.

period in individual cases, however, of course depends upon local conditions (and availability of funds).

For proper maintenance, a road must be remetalled either when (*a*) the metal has worn too thin, or (*b*) the surface has become too bad for traffic, and neither patch repairs nor continuous rut repairs will suffice.

The thickness of metal (above soling) for new roads is laid down in the general specifications, *viz.*—

Class I road 2 $4\frac{1}{2}''$ layers=Say 6″ when consolidated.
Class II road 1 6″ layer=Say 4″ when consolidated.
Road Metal. Class III road 1 4″ layer=Say 3″ when consolidated.

As the thickness of metal should not be allowed to get less than 2″ to 3″ thick, class II and III roads are likely to require remetalling shortly after completion, if they experience much traffic.

6. Almost the worst kind of metal is that derived from round boulders of varying kinds of rock. Such metal soon wears into pockets, and it should be avoided when possible.

Hard uniform limestone is easily consolidated, and gives a smooth surface which wears into continuous smooth ruts. It is rather dusty when dry, and produces a good deal of slippery mud when wet. It is peculiarly suitable for continuous rut repairs under Indian conditions of traffic.

Hard trap, granite, and sandstone (soft sandstone should never be used) requires to be broken smaller than limestone, and consolidated with the heaviest rollers available. Such material lasts longer, but wears into a nobbly surface, and is usually unsuitable for continuous rut repairs. It is less dusty when dry and less slippery when wet, than limestone.

Kunkar being a form of limestone, its characteristics are similar, but its wearing qualities under heavy traffic are poor, though its surface when in good condition is perfect.

Brick metal, obtained from vitified bricks, should not be used except in by-roads where only light traffic is to be expected.

It is *occasionally* necessary to use metal which without the addition of some kind of binding material will not consolidate at all. If the consolidation of such metal is carefully watched, it will be noticed that after a comparatively small amount of rolling, either wet or dry, the metal begins to move in waves in front of the roller, and the longer rolling is continued the more unstable it becomes. It would appear that the edges of the stone grind off into a fine sand, which acts as a lubricant. It is consequently necessary with such metal to use a layer of the best binding material available, which must be spread at the comparatively early stage, above referred to, when further rolling produces no improvement. It should be noted that binding material should not be mixed with the metal before rolling is commenced. The use

of metal with which earth has to be used for binding should be avoided, and it should not be used except under special orders.

The gauge of metal must vary with the hardness and nature of the material and the weight of the steam rollers available.

The normal gauge for stone metal is from 1½" to 2". As a general rule, the harder the material the smaller must it be broken. In certain cases, *e.g.*, using good quarried limestone or basalt and a 10 or 12 ton steam roller (which is the most suitable type) a gauge as large as 3" is practicable. Where the material available is laterite, and it is suitable and is used, gauges up to 4" are permissible. When boulder metal has to be used, the diameter of the boulders must not exceed 5", and the gauge of the metal 1½" or proper consolidation will be impossible. In such cases a binding material may have to be used, which should *not* be of loamy earth, unless absolutely unavoidable, and specially authorised in each case.

Care must be taken in drawing up specifications and contracts for the supply of road metal to provide that the metal shall be the best of its kind obtainable locally, that the maximum possible uniformity of quality is insisted on, and that, where quarries are concerned, metal obtained from long exposed surface blocks and boulders of small dimensions is specifically excluded as being inferior in quality.

Collection of Metal.

7. Except when impracticable, metal must be stacked absolutely clear of the berm proper. In the plains when space suffices, it should be stacked in continuous lines of uniform sections, as this arrangement greatly facilitates measurement. In hill sections it should be stacked in metal stacking places at suitable intervals (say 2 furlongs) specially constructed off the roadway.

Where it is necessary to have a gap, a stack of the same measurement as the gap should be placed as close to it as possible, and it is then an easy matter to see that the correct amount of metal has been collected.

On slopes it is necessary to take special precautions, and particularly to leave frequent properly sloped openings for side drainage, the absence of which leads to heavy longitudinal scouring of the road, and consequent rapid deterioration.

Metal should, as a rule, *not* be collected before it is actually required. Consolidation should not usually commence until all the metal required in the mile or portion thereof concerned has been collected and measured. Collection and consolidation should usually be carried out by full miles, except in cantonments.

Stacks of metal to be used for patching only should be maintained at specified intervals of about 1 furlong, and must be renewed to the dimensions laid down once or twice a year, a careful record of them being kept up. Patching metal must be stacked

well away from the regular berms where it will not interfere with traffic; it should also be on the side least suitable for stacking consolidation metal. Patching metal must be of the same kind as the road metal, and should be of gauge not greater than 1½″.

The gauge specified for road metal is the diameter of a ring through which each piece should pass in any direction.

Stone or brick metal should be passed over a ½″ screen *in situ*, the screenings being collected in heaps between the stacks and the edge of the road or stacking place. The screenings are used, supplemented by gravel, if necessary, for the top dressing. The rate for stone metal should include 10 per cent. screenings, in normal cases.

Road metal stacks should usually be 13″ high, counted as 12″ for the purposes of measurement, and of such a section as to give sufficient metal for the stipulated coat on the corresponding length of road. Firmas should be constructed accordingly, and checked periodically. Metal is collected on stacking places in hill sections in oblong heaps, the dimensions of which should be carefully checked, deducting 8½ per cent. for unevenness of ground, voids, loose stacking, and settlement, as in the case of roadside stacks.

Metal should be measured for payment in the stacks; soling usually when laid on the road.

Organization of work for metalling.

8. This is dependent upon—

(*a*) The arrangements possible for traffic during metalling.

(*b*) How many rollers can be employed together.

If possible, all traffic should be diverted off the portion of the road being metalled; but if necessary to meet traffic requirements, in the case of Class I roads metalled 16′ wide or over, or occasionally in the case of Class II roads metalled 12′ wide, half the width can be remetalled at one time. The former is the normal method. When the latter method is adopted, special care is necessary to ensure continuity of correct camber.

Frequently in hill sections, and sometimes in short sections in the plains, the traffic cannot be side tracked or confined to one half of the road. In such cases the various operations of consolidation have to be condensed and concentrated, and the work finished as rapidly as possible, engineering considerations being subordinated to traffic requirements.

Process of metalling.

The process of remetalling consists of the following four operations (the first operation is omitted in the case of new metalling):—

(1) Scoring the old surface.

(2) Spreading the new metal.

(3) Dry rolling.

(4) Watering, wet rolling, and surfacing.

Steam roller output.

The "output" of a roller is the number of cubic feet of loose metal which it can properly consolidate in a working day. The

corresponding length of road completed (which depends on the thickness of the layer of metal loosely spread and the width of metalling) is exhibited in the following table:—

Lengths of roads completed daily by one roller consolidating a $4\frac{1}{2}$ inches layer of metal.

Width of metal.*	OUTPUT OF ROLLER IN CUBIC FEET.		
	700	600	500
Feet.	Feet.	Feet.	Feet.
16	117	100	83
12	156	133	111
9	207	178	148

NOTE.—An output of 700 cubic feet is attainable with good limestone by a 10-ton roller doing an honest 8 hours' day.

Working with one roller only, the length of road closed to traffic should be three times the length which can be consolidated by the roller. The first third will be scored, the middle third spread with loose metal, and the last third consolidated.

Separate gangs should be employed whole time on each section and should advance daily one stage ahead.

Working with two rollers grouped, the first will carry out dry rolling and the second wet rolling and surfacing, so that progress will be doubled, and the length of road isolated will be twice as great.

Additional separate gangs should be employed for preparing and maintaining diversions well in advance, and for making up berms immediately in rear of the last roller. This latter process should not be left till the whole mile has been consolidated.

Whether work is done by departmental agency or by contract, it is equally important to insist on its organization on the above lines. In no circumstances is it permissible to extend the operations of scoring and spreading beyond the limits required for one day's work of the roller or rollers.

The advantages obtained by grouping rollers in pairs, as advocated, are as follows:—

(i) Each mile is finished in about half the time with less inconvenience to traffic.

(ii) Supervision of work is easier and superior results are obtained.

(iii) Supplies of coal, water, and stores for the engines are facilitated.

(iv) Mechanical supervision of rollers is improved with resulting increased efficiency, particularly if it is possible to

attach a mechanical mistri to each group of rollers, an arrangement which will usually reduce the number of idle roller hours and pay directly, by giving an increased output per roller.

When the road cannot be closed to traffic during metalling the following special arrangements will be necessary.

(i) Scoring and spreading must be confined to a short length, in front of the leading roller.

(ii) The metal must be spread as evenly as possible from the side stacks in one operation, and must on no account be piled in heaps on the roadway preparatory to spreading.

(iii) The road gangs must be ready to assist traffic by all means in their power.

(iv) The roller driver must be ready to pass his roller once or twice over the roughly spread metal if asked to do so by a motor driver, and in any case must get his roller to one side without delay.

(v) Three rollers grouped should be used, the function of the third being to reroll the completed work, and to pay special attention to patches showing signs of failure, which are to be expected owing to the hastily completed consolidation being subjected to immediate traffic.

(vi) Extra good supervision and organization are necessary to prevent scamped work, waste of time, and blocking of traffic.

Surface Scoring.

9. The old surface must be freed from mud or dust, and well roughened with pickaxes, and all considerable hollows must be filled with metal, before the spreading gang is permitted to commence work. The full working width must be demarcated with picks working along properly aligned strings.

If, as frequently happens, the barrelling has disappeared, it must be restored as much as possible by well picking up and relaying the surface of old metal to camber. In bad cases, scarifying is desirable.

Spreading Metal.

10. Earthen bunds, puddled on the inner side, and of the height of the metalling, should be made along the edges of the area to be metalled, in order to prevent the metal from spreading.

The metal should then be raked off the stacks into baskets and spread to correct camber and thickness on the road. This cleans the metal from fine stuff and dust.

In straight or nearly straight sections it is absolutely essential to use properly made full width gauges or templates fitted with a central plummet, the closer the better, but certainly not more than 50 feet apart. The depth of the plank forming the gauge should be 4½ inches (or the thickness of the metal coat), so that when the

metal has been properly spread the gauges are buried just flush with the surface. The intermediate work is then easily tested with a cord stretched between gauges, which latter are not to be removed until the spreading has been passed at the end of each day's work.

On curves similar gauges are required, but with straight profiles in place of cambered profiles. Sets of these for different cross slopes (1/7, 1/10, 1/15, 1/20 should suffice) are required, and each must be clearly marked with its slope.

The transition strips between cambered sections and banked curves must be spread by eye as gauges cannot be used, but in all cases the edges to the metal must be clearly defined by lines.

11. Rolling should be commenced from the edges and continued towards the centre of the road. Rolling.

The stage when watering must be commenced varies with the nature of the stone, and as water is often expensive, care should be taken that it is not applied till necessary. The local specifications should specify the number of roller passes to be given dry, for each kind of stone according to the weight of roller used, on the basis of correct data obtained from experience.

The first principle of macadamizing is thoroughly to consolidate clean square evenly graded stone metal. Therefore no binding or surfacing material may be used till consolidation is complete (except as mentioned in paragraph 6).

The number of roller passes required depends on the weight of the roller, the nature of the metal, the amount of water available and the skill of the engine driver, but it is the worst kind of economy to cut the rolling fine.

One of the main causes of poor output is the laziness of the drivers and the difficulty of ensuring continuous honest supervision of their work, and the most important factor in settling their pay is their average output of good quality work. The other factor is the care they bestow on their engines. It pays to give a considerable increase in the wages (either directly or by an "output" allowance) of any driver who is really good in both respects. From this the necessity of keeping accurate records of working days and output is apparent.

The camber and banking templates or gauges mentioned in paragraph 10 must be systematically used during consolidation to check the surface.

During military operations, in cases where water is unobtainable, fair temporary results can be obtained with limestone (and possibly with other kinds of metal that have good binding properties) by dry rolling only, then spreading a thick layer of shale or bajri, and leaving the traffic to do the rest. The binding material should not be mixed with the metal before rolling.

Surfacing. 12. After thorough consolidation, the screenings, supplemented as necessary by screened gravel or coarse sand previously collected should be spread evenly, watered and rolled. The object is to fill the interstices between the metal and produce an even water-proof surface. The less the material used to obtain the des ired resultthe better, and an average thickness of ½ inch is usually ample. Earth must *never* be used for this purpose. When kankar, laterite, or brickmetal is used, it should be rammed in the first instance with iron rammers, or rolled with only a light roller (bullock roller or 6 ton steam roller) until consolidated. Care must be taken in all such cases that the roller used is not so heavy as to crush the metal.

Maintenance of newly consolidated metal. 13. Newly consolidated metal must be kept under careful observation for some little time, so that any places which work loose may be at once repaired, as otherwise the damage will very quickly spread and extra expense will be involved (See also paragraphs 17 and 18).

SCARIFYING METHOD OF REMETALLING.

Scarifying. 14. This is an economical method of resurfacing old roads, but should not be attempted unless the average thickness of metal over the soling is at least 6 inches. It is particularly useful in localities where berm material is expensive, as the road surface level is not raised by remetalling by the scarifying method.

A suitable method of resurfacing by the scarifying process, based on experience on the Jhelum Valley road, where excellent results have been obtained, is described below :—

The road in question has 12 feet of metal, and is subjected to exceedingly heavy traffic which cannot be side tracked. The scarifier is made by Messrs. William Thackeray and Sons, Malton, Yorkshire. It is fitted with four teeth or tynes of special steel which wear out rather rapidly. Messrs. T. Pratt & Co., 304, Bow Bazaar Street, Calcutta, sell a brand known as "Violet mark treated steel" which has been found quite satisfactory for replacements.

The scarifier is towed by a roadroller of not less than 8 tons weight by a strong steel cable some 20 feet long. The teeth should be set to cut not more than 2" deep below the road surface, as this actually loosens at least 3 inches of metal. On the flat or down hill all four teeth are used, but up steep inclines only two of the teeth should be put in action, to save excessive wear on the driving pinion of the roller.

The scarifier is easily steered by an ordinary cooly, who finds no difficulty in keeping a good line and obtaining uniform results. The only precaution necessary is to drop the teeth *gradually* when

commencing a cut to save sudden strains, but no difficulty has been experienced in training the cooly accordingly.

After completing a cut the cable is detached from the engine, which then backs into position for the reverse cut. The steering handle of the scarifier is set over at right angles, the cable being attached to the other end of the engine, and the latter then tows the scarifier round. When the line has been picked up again the cooly gradually engages the tynes. The operation of reversing is very simple and quick.

The method of working is as follows:—

On beginning work the engine driver scarifies a length of 50 feet of road, and then proceeds to roll to a finish his previous days' work of 2 to 2½ chains in length. A gang of not more than 15 men rakes the newly scarified surface to the side to the road, and then back again to the 12 feet alignment, thus removing all fine stuff and dust to a large extent. The old metal is then well watered (which cleans it still further), and a layer of fresh metal *of the same kind* is spread 1½ inches thick over the surface, equivalent to one-third of a full coat. The engine scarifies a further 50 feet of road, and the process is repeated at intervals till the full length of 2 to 2½ chains has been prepared ready for next day's consolidation. If the scarified metal is found to be so bad that screening is necessary instead of raking the gang must be increased accordingly, or the output will be reduced to some 1½ chains a day.

Thirty days only are allowed for scarified consolidation of one mile, including a margin for all delays, and the work is given out on contract with a penalty for exceeding this time. The cost is worked out at Rs. 52 per mile for scarifying only, as against Rs. 146 for similar work done by hand.

Fresh metal *of the same kind* should always be mixed with the old metal, to the extent of 33 to 40 per cent. The life of a mile thus treated is about ¾ of the life of a mile given a full new coat.

Rut Repair Method of Remetalling.

15. As another alternative to complete remetalling, this process is particularly applicable to narrowly metalled roads (*e.g.*, 9') surfaced with limestone, which have worn into clearly defined ruts under traffic. Miles so treated can hardly be distinguished from newly consolidated work, and should have a life nearly equal to that of a complete 4½" renewal coat. The resulting repairs stand occasional heavy motor lorry traffic satisfactorily. **Continuous Rut Repairs.**

A continuous trough should be excavated over each rut by hand, with side slopes of about 60°. The old metal should be raked into heaps at the side of the road, and the troughs filled with new metal *of the same kind* spread 4½" deep. The old metal (the

quantity of which is not much) is then spread on the new metal, having been fairly well cleaned by the raking process. The whole is then well watered and consolidated with a roller in the usual way. No extra binding material is required, as the old metal answers the purpose.

The 'output' of a steam roller on this class of work should not be less than normal. Rut repairing has been done in the N. W. Frontier Province, at Rs. 10 per 100 cubic feet, measuring the full section of the excavated trough.

The economy of this method is apparent. Repair work of this kind should be done by the mile, and should not be attempted by hand ramming instead of rolling, as opposed to pot-hole repairs.

Road Rollers.

Steam Rollers.

16. Instructions regarding the working and maintenance of steam road rollers are given in Volume IV (Electrical and Mechanical and Water Supply).

Maintenance of Road Surface.

Patching.

17. The greater the cost of road metal the more should be spent on patching (*i.e.*, ramming in metal with water and *clean* bajri, into pot-holes which should be cut out first), and on other measures such as periodical spreading of shale or bajri, in order to increase its life.

There is generally a tendency to spend too much of the repair grant on remetalling, and too little on patching.

Too much stress cannot be laid on the careful and systematic patching of roads, as it is often common practice to neglect patching till a mile has got into noticeably bad condition, instead of patching newly consolidated miles at the first signs of local failure, and so maintaining them in good condition. More care, supervision, and money spent on patching miles before they get really badly worn means a very appreciable increase of life, and a consequent great saving in the annual cost of remetalling.

Patching metal must be *of the same quality* as that used for remetalling. Patching is the normal and most important duty of the regular road gangs, whose strength per mile depends largely on the amount of such work to be done. Red flags should always be used on either side of a patching gang to warn motorists, and the men themselves must be trained to signal which side the motor should pass any patch not yet filled after excavation. They must never leave their tools lying about on the road.

The gauge of patching metal should never exceed 1¼", and should be less when the stone is very hard.

18. The systematic spreading of bajri on the metalled surface of a road, by the road gangs, directly the individual stones of the metalling begin to show up, stops the metal picking up and prolongs its life. This is particularly important in the hot dry season of the year. Earth must *not* be used for this purpose. Spreading bajri.

The bajri should be clean river bajri, shale, or sand, and it should not be laid more than ½ inch thick. Watering in addition is also very desirable in hot dry weather, and rolling in bad cases particularly when there is little traffic or when the surface has started to pick up extensively.

19. When a road is called upon to meet exceptionally heavy traffic, regular and systematic surfacing, as described above, is essential. Thatching grass can be used as a temporary emergent measure instead of bajri, but it is less effective and requires more frequent replacement. Surface protection under exceptional traffic.

20. The stipulated camber and super-elevation must be carefully maintained in repairing berms and metalling. In all cases in which a road is below standard in these respects, it must be brought up to standard when repairs are carried out. Camber and Superelevation.

Special Methods of Road Surfacing (Oiling, Tarring, and Concreting).

(See Chapter II, paragraph 40.)

21. To promote durability under heavy traffic, and to minimize dust and mud, the ordinary water bound macadam may be surfaced or replaced by the following methods :— Special methods of road surfacing.

(i) Surface Oiling on water bound macadam.
(ii) Surface Tarring on water bound macadam.
(iii) Surfacing with Asphaltic oil or composition.
(iv) Metalling or remetalling with Tar macadam.
(v) Metalling or remetalling with Bituminous or Asphalt macadam.
(vi) Surfacing with Pitch-grouted macadam.
(vii) Cement concrete or reinforced concrete surfacing and foundations, in lieu of metalling and ordinary soling.

The British Road Board Standard Specifications relating to (ii), (iv) and (vi) are given in Appendix XLIX for guidance, and supplementary remarks regarding these and the other methods are made in the following paragraphs. The camber of the road surface should normally not depart from the normal standard of 1 in 40, for metalled roads.

22. Oiling is the simplest and cheapest method of treating water bound macadam. It is in use on subsidiary roads in the United States and elsewhere, and has been used successfully at Delhi and other stations in India. Oiling water bound macadam or earthen roads.

United States experience shows that oiling greatly improves kutcha roads, provided that the surface is dry and properly made up to camber, and rolled before treatment. In this connection it is of value for treating the earthen berms of metalled roads in India, under the same conditions. In the case of earthern berms or road surfaces, however, oiling will of course require comparatively frequent renewal. Where there is very heavy rainfall, the durability of oiling on earthen surfaces, and its consequent practicability, are of course, considerably lessened.

Tar oiling comprises the pressure spraying of the surface with a suitable number of coats of tar oil or petroleum oil with an admixture of 5 per cent. coal tar.

The following conditions must be fulfilled :—

(i) The surface must be dry.

(ii) The surface must be clean, *i.e.*, all dust must be well swept off immediately beforehand, and all depressions, etc., repaired and filled up.

(iii) Traffic must be kept off the portion under treatment until the oil has been properly absorbed, and subsequently coated, usually, with a thin top dressing of fine gravel or clean sand.

(iv) The oil and tar, which do not remain in combination, must be mixed at the time of application, or alternatively kept well mixed throughout.

(v) The oil should be of the consistency of thin golden syrup, the tar that of treacle.

(vi) The mixture must be kept free of any foreign matter.

(vii) The treatment must not be carried out during, or in proximity to, the rainy season.

The following desiderata are advocated for efficient and economical work :—

(i) The mixture must be applied hot (this is essential where oils containing 40 per cent or more of asphalt are employed), where necessary to promote the suitable liquidity.

(ii) A travelling pressure road oiling machine, with heater attachment where necessary, should be used. (The machine manufactured by the Austin Manufacturing Company of Chicago, which can be drawn by a pair of horses or bullocks, or by a lorry, is very suitable.)

(iii) A mechanical broom should be used for efficient sweeping. (The Austin Company manufacture a suitable street sweeper of this type, which can be drawn by a pair of horses or bullocks, or a lorry, for use in conjunction with the travelling oiler).

As in the case of all other special road surface treatments, the specifications and the results depend upon the local climatic conditions, the nature of the metalling, and the traffic.

A description of the method adopted at Delhi, using 3 coats of fuel oil with 5 per cent. of ordinary coal tar, is given in Appendix L. In this case, the oiling requires renewal annually.

The British Road Board Specifications for Tar and Tar Oil are given in Appendix XLIX, *viz.*:—

Specification No. 4, Tar No. 1.
,, No. 5, Tar No. 2.
,, No. 6, Tar oil.

Tar No. 2 is most suitable for hot dry climates.

Patch repairs to oiled metalled roads should be done with tar and clean metal, and if the work is efficiently done, and renewed at due periods, watering to allay dust should not be necessary.

Asphaltic oils and compositions can also be used for oil spraying or surfacing, and their use should be studied.

Tarring water bound macadam roads.

23. Surface tarring on water bound macadam roads is more durable and efficient than oiling, but more expensive. Surface tarring or spraying consists in the application of hot coal tar (one coat or more as necessary) to the metalled surface, the surfacing being finally covered with a thin layer of fine gravel or clean sand.

This method has been used with varying degrees of success in various parts of India, notably in Bombay, Madras, Calcutta, and Lahore.

Surface tarring has the disadvantage of promoting a slippery surface after rain (apart from the softness caused by great heat, an ever present difficulty in the summer everywhere, which is particularly applicable to hot stations in India). Roads so treated must therefore not have an excessive camber. With certain kinds of metal, *e.g.*, in the case of limestone as used at Lahore and Rawalpindi, the surface becomes insuperably slippery for animal traffic, and if tarring is to be done in such cases more suitable metal must be used to overcome this disadvantage.

General directions for surface tarring on water bound macadam, and specifications for tar, are given in the British Road Board Specifications in Appendix XLIX, *viz.*:—

Specification No. 1, Surface tarring.
,, No. 4, Tar No. 1.
,, No. 5, Tar No. 2.

(Tar No. 2 being most suitable for hot dry climates).

The conditions given in paragraph 22 for successful oiling are of equal application in the case of tar surfacing or spraying: in this case it is essential for the tar to be heated to boiling point (about 220° to 250°F) when applied. The use of a travelling

sprayer and heater, and of a mechanical broom, as mentioned in paragraph 22, is equally desirable for efficiency and economy.

The Punjab Specification for tar spraying is given in Appendix L.

Where there is heavy traffic, two coats are necessary, the second two or three months after the first coat.

The tarring should be renewed annually except on roads carrying light traffic, where it must be renewed as necessary. Practical experience in India shows that efficient surface tarring will stand under regular light traffic up to 3 or 4 years.

Patch repairs to tarred roads must be done with clean metal and tar.

Tar Macadam.

24. The use of tar macadam instead of water bound macadam, in renewal coats or in initial metalling, is greatly preferable to either oiling or tar surfacing, but it is correspondingly more expensive in initial cost, although in upkeep it should in the long run prove no more expensive, provided that the specification is suitable to the climate.

Under very heavy traffic, tar macadam is the only method to use, as compared with oiled or tarred water bound macadam, where its initial cost can be afforded.

Tar macadam, similarly to tar surfacing, has been used with varying degrees of success in various parts of India, and is now adopted as a standard in certain towns where a satisfactory specification suitable to the local conditions has been arrived at by experience. It has been found that the climatic difficulties can be eliminated by adherence to specifications.

In this method, the broken metal, which must be perfectly dry, is first coated with hot tar, and is subsequently laid on the road and rolled in, less consolidation being required than in the case of water bound macadam. It is desirable to apply a coating of boiling tar to the surface after the road has been used for traffic for several weeks.

Good and suitable road metal is necessary, equally with the use of tar of suitable quality; in this connection, apart from the necessity for good wearing qualities, limestone is found to promote an undesirably slippery surface when used in tar macadam, as in the case of tar surfacing.

The use of a tar matrix is particularly advantageous where otherwise good metal with bad binding properties has to be used.

General directions for the use of tar macadam, and for tar, are given in the Standard British Road Board Specifications in Appendix XLIX, viz.:—

Specification No. 2, Tar macadam.
No. 4, Tar No. 1.
No. 5, Tar No. 2.

(Tar No. 2 is the usual specification, particularly for hot climates.).

At Lahore good results have been achieved by using ordinary gas tar heated up to 200° F, with 25 per cent. of pitch added; in this case it was found necessary to keep the metal (for 2 or 3 months), until the coating became "tacky," as laid down for Tar No. 1, in the Road Board Specifications, before laying and consolidating.

Self-contained portable machines for the preparation of tar (and bituminous) macadam, where such work is to be done on a large scale, are manufactured. These machines dry the stone metal, heat the tar (or bitumen), and mix them together in the proper proportions, discharging regular quantities of properly coated metal. They can be driven by a steam road roller or tractor or other power unit of similar H. P.

25. A specification for the surfacing of a road with bituminous macadam (taken from "Specification" published by Technical Journals Limited, London), is given in Appendix LI. Bituminous or Asphalt Macadam.

This is a modern method of surfacing which should be studied. Tar macadam mixing plant (see paragraph 24) is used for such work on a large scale.

26. General directions for surfacing with pitch-grouted macadam, and specifications for pitch, are given in the British Road Board Specifications in Appendix XLIX, *viz.*:— Pitch grouted macadam surfacing.

Specification No. 3, Pitch-grouted macadam surfacing.
,, No. 6, Pitch.

This method is not so satisfactory as the use of previously mixed tar macadam.

27. Cement concrete, or reinforced concrete, can be used instead of soling for the foundations of tar macadam or water bound macadam roads, (in addition to its normal use in paved roads, which are not dealt with in this Handbook), where owing to the heavy weight and speed of the traffic and the soft nature of the subsoil increased strength in foundations is necessary. Cement concrete road foundations.

When used in macadam surfaced roads, the concrete is finished with a toothed surface to form a key for the surfacing material.

Particulars of cement concrete and reinforced concrete road foundations (taken from "Specifications" published by Technical Journals Ltd., London), are given in Appendix LII.

28. The use of cement concrete or reinforced concrete road surfacing has been largely developed on highways in the United States, and is extending elsewhere. The durability and cleanliness of this type is obvious, and although its initial cost cannot be afforded at present, it should cheapen with the development of modern methods of work and of cement manufacture in India; its details and development should therefore be studied, for adop- Cement Concrete road surfacing.

tion when and where practically and economically suitable. With cheap cement, efficient plant, trained labour, and materials ready to hand, the first coat is not unduly expensive, and maintenance costs are of course comparatively small.

Particulars of cement concrete and reinforced concrete road surfacing (taken from "Specifications" published by Technical Journals Limited, London) are given in Appendix LII.

A method of asphaltic surfacing, used successfully on reinforced concrete bridge decking in the N. W. Frontier Province, for the purpose of water proofing and hardening the surface of the concrete for traffic, is described in Chapter VI, paragraph 47.

Notes on concrete road design and construction for India are given below.

Concrete road design and construction for India.

29. As regards design, assuming a sound foundation, a subject which is dealt with later, the first question is the width of the road to be adopted. At first 14 feet was considered sufficient but so great became the increase of traffic when the advantages of the concrete road were realised that eminent engineers have had to alter their views and it is recognised that a width of 18 feet is the best standard practice.

Thickness.

In certain climates a uniform thickness of 6 inches for medium traffic and 8 inches for heavy traffic can be adopted as a safe standard, but in a hot climate other factors come into play and after spending large sums in experimental work it has been proved that the following action will take place. When the temperature rises the surface of the concrete is heated first and therefore expansion tends to take place at a greater rate near the surface than below, causing the concrete slab to curl up at the sides; *vice versa*, on cooling at night or in cold weather, the contraction tendency is again greater at the surface with the result that the concrete slabs tend to lift at the centre. Under these conditions it was found that lorries travelling with the near wheels about 2 feet from the edge of the concrete would in time break the concrete at that distance from the edge. To meet this problem various designs were submitted to further heavy tests and it was found that the best form of construction for such conditions was a concrete slab 9 inches thick at the edges tapering to 6 inches at a distance of 2 feet from the edge, and this allowed of reducing the whole of the mid portion of the slabs to 6 inches thick even for heavy traffic. A longitudinal bar $\frac{3}{4}$ inch in diameter should be provided along each edge of the concrete slabs, and a longitudinal joint is recommended along the centre.

It is considered that when a longitudinal joint is made, a bed about 6 inches wide and 3 inches deep of 8 : 1 concrete should be laid ahead of the work, so that as the joint works there is no chance of foreign material rising through the joint from the sub-base—

the joint is thus kept clean and solid. In this type of construction no reinforcement other than ¾ inch bar is necessary unless the foundation is bad.

It is naturally essential that the stone or gravel and the sand should all be free of any foreign matter, such as sulphur, which would be injurious to the cement, and whatever is used must be clean. The stone should be graded from 1½ inches to ⅜ inch and the sand should all pass through a ¼ inch square opening and not more than 10 per cent by weight shall pass a sieve having 50 meshes per lineal inch. If only a fine sand is available, it will be found that a greater quantity of cement must be used to ensure every particle of sand being covered with cement. Unless this precaution is taken the concrete will fail. If the concrete is hand mixed, it is recommended that the sand and cement be mixed dry before adding the dry coarse aggregate and then the whole mixed wet. Materials.

The water should be clean and, again, in a country like India, it should be analysed to ascertain whether anything is present likely to be injurious to cement and if this is the case a very simple treatment will probably eradicate the evil.

The cement should be of the highest quality and slow setting, the difference between quick setting and quick hardening is not always fully realised. A slow setting cement may be quick hardening and if this is the case it is an advantage in road construction.

Generally speaking the proportion of aggregate to cement should be 4 parts of coarse material to 2 parts of fine material to 1 part cement (4 : 2 : 1).

In towns it is sometimes preferred to make the road in two courses, the upper being richer to withstand the concentrated wear and tear and in which case the top 1½ inches depth should be composed of 2 parts ½ inch to ¼ inch chippings or crushed ballast to 1 part sand to 1 part cement (2 : 1 : 1) or not weaker than 3 : 1½ : 1. Should this method be employed the top course must be laid almost at the same time as the lower course and always before the initial set has taken place in the lower course.

In view of the fact that these roads allow of high speed it is advisable to bank the curves in proportion to their radius, the maximum super-elevation should occur through the centre half of the arc and should taper off for a distance along each straight equal to a quarter of the length of the arc.

In Great Britain these roads are often constructed in alternate bays about 20 feet wide, the intermediate bays being filled in about 14 days afterwards when the initial contraction of the concrete of the adjoining bay has taken place, otherwise it appears advisable to have expansion joints at intervals not greater than 70 feet apart; these joints, about ½ inch wide, should be filled with a fibrous elastic waterproof material.

Foundations. It is not fair to expect the concrete to do everything, neither is it fair to expect it to carry loads over an indefinite length, and it is necessary therefore to examine the foundation. If this will stand an eight-ton roller it is good enough and all that is advisable is to blind it with a thin layer of sand or chippings to ensure the concrete being kept clean when deposited. If the foundation is liable to get spongy in wet weather and is then weak it should be properly drained by cutting diagonal trenches at intervals about 18 inches in width and depth and filling with gravel or some material which will allow the water to drain off and thus keep the top 18 inches fairly dry.

If the foundation is liable to expand and contract a few inches of ashes rolled in will act as a cushion between the material and the concrete and thus prevent the latter from cracking.

All banking must be thoroughly consolidated and when the materials for filling are not all homogeneous they should each be deposited in thin layers and not promiscuously. The use of a ten-ton roller is advised on banks.

Construction: After grading the sub-base the forms should be set to the correct grade and made absolutely firm as the tamper will work on them later. These forms should be at least 2 inches thick. It should be remembered that if these forms are not firmly butted it is probable that a ridge will show in the finished concrete surface.

Before depositing the concrete the sub-base should be wetted in order that the latter may not absorb any moisture from the former.

The concrete should be mixed to such a consistency that if stamped on half a dozen times with a booted foot, it just shows a moisture on the surface; the concrete is only weakened by making it wetter than this. About 5 feet of concrete should be deposited and tamped and the work followed up accordingly. As each 5 feet or so is completed it should be immediately covered up with canvas, care being taken that the surface is not disturbed. The canvas should be kept damp. The canvas, which may be in frames if desired, should be removed the next day and the concrete covered with sand or palm branches and kept wet for ten days.

The tamper for a road 18 feet wide may be made out of 10 inches by 2 inches, or 9 inches by 2 inches timber according to the amount of camber—if 9 inches by 2 inches is used a piece 2 inches square should be added at the top to prevent warping—this piece being laid on edge and nailed down.

The tamper should be provided with a shoe of wrought-iron $\frac{1}{2}$ inch thick and, say, $2\frac{1}{4}$ inches wide which should be screwed in. The camber may be 1 in 48 and the tamper for a road 18 feet wide would be 19 feet long, the 6 inches at each end being level to so

as a bearing on the forms. The handles should be close to the end.

Many engineers prefer the side forms to project an inch above the level of the finished surface at the edge of the road so that the concrete does not spew over the form as this would make it necessary to continually clean it away from under the tamper where the latter bears on the forms.

Other points arise in the construction of a concrete road but in the above are given the main points which if followed out carefully will give an excellent road. General.

Maintenance of Berms.

30. The minimum width of berms to be maintained is laid down in the road specifications, and, unless absolutely unavoidable, these widths must never be encroached on by stacks of metal. Berms.

Where a road is unbanked, or is on a very low bank, there is usually a wide space between the limits of the regulation berms and the lines of trees or borrow pits, and it is important to maintain these outer berms also in good order. For this reason the banked edges of the regulation berms should slope off gradually to the outer berms, so that wheeled traffic can make use of the full width of the road area when necessary. This also implies that earth for making up berms must only be taken from borrow pits; digging holes in any other places must be strictly prohibited, and excavations must be filled in when they already exist.

A good berm should be such that in fair weather a motor car can traverse it without inconvenience or risk of severe jolts at a speed of 20 miles per hour or more. If it fulfils this condition in dry weather, it will usually (if given a sufficient slope) be safe to use at slower speed in wet weather also.

Slopes of berms must be within the limits specified in the general specifications for roads (Chapter II).

Berms must be consolidated with hand rollers or by efficient ramming, especially when such making up is necessary. Otherwise not only are they unusable by wheeled traffic for a long time after remaking, but also, in the event of heavy rain, they act like a sponge, and the water they hold is apt to penetrate under the soling and cause local sinkage.

Rain cuts must be promptly repaired.

On steep gradients, where carts like to use the soft berms as brakes, and where the berms consequently get badly cut up, they can only be kept in good order by the use of shingle or rough metalling of some kind.

The growth of grass is to be encouraged.

On no account must berms be level with or higher than the road itself, even when first made. Good drainage from the road outwards is essential.

The cases in which berms are to be soled are mentioned in the general specifications for roads (*vide* Chapter II).

Money is often wasted and good berms are spoiled, by following the routine practice of dressing them to a specified width immediately after remetalling. An estimate for remetalling must include only what is *actually* necessary for the mile in question to keep the berms within the proper limits of slope, and a stereotyped item for berm dressing must never be included. When remetalling with scarifying, little or no berm dressing is usually necessary, as the surface of the metal is not raised.

Apart from the berm dressing allowed for in remetalling estimates, all berm repairs should be carried out by the regular road gangs, and usually it is only when the berms have been allowed to fall into disrepair by long neglect that any special repair expenditure on them is legitimate.

Gauges. To keep berms in proper shape, or to effect extensive repairs to them, the use of gauges made to specified slopes is essential.

MAINTENANCE OF DRAINAGE.

Necessity for improving drainage on existing roads. 31. The construction of a road along a hillside alters the whole system of natural drainage, and improvements to the drainage are generally necessary as a result of the first rainy season after construction. Improvements to drainage should be regarded as an important duty; they include providing additional culverts, etc., (as new works) in addition to improving drains.

Side drains. 32. Side drains on all roads are seldom big enough in the first instance, and can only be kept in order by constant attention. A good road in hilly country will soon be ruined during the rains, if incessant attention is not given to clearing the side drains and removing small slips as soon as they occur.

For this reason the strength of road gangs should generally be increased during the rainy season, on hill sections liable to slips.

In the plains it should be seen that water does not lodge between the foot of the road bank and the side drain, but that sufficient outward slope is given right across the whole strip of land between road boundaries. On very flat irrigated lands special care must be taken to provide proper outlets and well graded drains of sufficient capacity, so as to prevent waterlogging of the subgrade.

For the same reason the side drains must never be used as irrigation channels, and encroachments of this kind must be stopped at once.

In certain classes of alluvial soil side drains sometimes develop into deep cuts or small ravines, which threaten the existence of the

roadway. One way of dealing with such cases is to divide up the watercut into sections of suitable length, and form each section into a gentle slope (say 1 in 100) followed by a sharp drop, which must, of course, be revetted like a dry stone retaining wall with foundations taken several feet below the next step. The top surface of the wall should be given a fairly sharp dip to confine the flow to the centre and discourage cutting round the sides, and in addition for some distance immediately above the crest of the wall the ground must be pugged or well rammed to prevent water getting down behind it. The foundations of the wall should be protected with a cushion of large loose stones sloping upwards till they merge into the next flat portion or step, thus forming a water cushion. The fact that such treatment entails heavy expense points to the necessity of taking very early action to prevent cuts from getting serious. (See also Chapter VIII, paragraph 15.)

33. Side drains, catchwater drains, culverts, catch pits, scuppers, and causeways must be systematically cleared by the regular road gangs. Special arrangements for extra labour are usually necessary on hill roads after heavy rainfalls, but except in such cases the regular road gangs must be made responsible. Clearing drains, culverts, etc.

Maintenance of Road Structures and Road Signs.

34. All road structures, particularly including river protection works, must be systematically and periodically inspected, and emergent repairs carried out without delay. Road structures and Road Signs.

Lack of attention to systematic supervision of this nature leads to such incidents as culverts collapsing under traffic, causeways being scoured out and destroyed, and bridges being rendered unstable.

Such inspections must be made at least at the commencement, middle, and end of the rainy season.

Direction and warning signs, milestones, etc., should be annually repainted when funds permit, and emergently repaired at other times when essential. Danger signs in particular must always be kept in good order. Steel work on bridges, etc., must be periodically repainted.

Petty repairs to and renumbering of culverts, etc., should be similarly done annually.

Road Gangs.

35. The chief duties of road gangs are :— Duties of Road Gangs.

(1) To remove all loose stones, glass, nails, horse-shoes, etc., from the metalled roadway on their way to and from work. This is most important on all motor roads, and must be insisted on.

(2) Patch repairs.

(3) Clearing drains, culverts, causeways, etc.

(4) Upkeep of berms.

(5) Surfacing.

(6) Removing slips.

(7) Petty repairs to dry stone walling, etc.

(8) Reporting to the Sub-Overseer in charge of their section any damage to culverts, walls, etc., which they cannot repair and particularly any unauthorised tree cutting.

Composition of Road Gangs.

36. A road gang consists of a mate and a certain number of coolies, whose beat should not usually exceed a section of 6 miles. The strength may vary from a minimum of one cooly per mile upwards, according to local conditions, but it is essential that the gang shall work in a body under the mate, that their duties shall be prescribed daily or weekly by the Sub-Overseer, and that the latter shall measure up their work regularly and enter it on the muster roll.

Each mate should always be held responsible for the efficient maintenance of the section of road in his charge. A combined system of rewards and fines goes a long way towards achieving efficiency. Punishing individual road coolies is of little use: the mates should be dealt with.

A regular equipment of tools and plant (rammers, pickaxes, shovels, screen, gauges, and hand roller for berms), should be provided for each gang, and held in charge by the mate, and where necessary a donkey with pakhals or tins should be attached to each gang for carrying water for road repairs.

Malis.

37. For the upkeep of roadside trees, malis should be employed in charge of definite sections of road, with attached water donkeys only if wells or irrigation water, etc., are not near the road. They should be held responsible for the trees and roadside grass within the road boundaries in their sections, and for reporting at once all cases of wilful damage beyond their control. Records should be maintained of the approximate numbers and descriptions of trees in their charge, including particularly young trees, in order that their work may be properly supervised; a system of rewards or fines according to results is effective in their case, equally with the mates.

Badges.

38. All regular road maintenance personnel, including malis, should be provided with distinguishing badges. A suitable system is for the mates and malis to wear a M. E. S. Chuprass with appropriate designation, and for mates, malis, and gangmen to wear a coloured (*e.g.*, red) pagri.

MAINTENANCE OF BOAT BRIDGES.

39. Boat bridges and ferries on Indian roads are maintained in accordance with local specifications and rules, a special establishment of boatmen, etc., being permanently employed at each bridge. Boat Bridges

A general description of Boat Bridges is given in Chapter VI, paragraph 41.

Attention must be given to the following points :—

(i) Proper maintenance of anchorages, superstructure, and boats, and prompt replacement of defective parts.

(ii) Provision and upkeep of warning, traffic capacity, and speed limit road signs (see Chapter IX).

(iii) Provision and upkeep of gate barriers, and red lamps, for use at night and when the bridges are unavoidably closed for traffic owing to repairs, etc.

(iv) The perpetual presence on the bridges of duty boatmen, and watchmen at night.

(v) The annual replacement of portions of the boat bridge equipment, according to the life and wear of the various parts.

TRAFFIC CONTROL.

40. Adequate barriers and warnings must *always* be provided at the ends of stretches of road under repair. These must be lit at night by red lamps, and watchmen posted at them (see Chapter IX, paragraph 4). Barriers and Diversions.

Care must be taken that suitable diversions for traffic are provided, when the road is blocked for repairs, and that they are properly maintained.

41. All breaches on roads due to floods, etc., which cause a stoppage of through traffic, must be at once reported by the Assistant Commanding Royal Engineer to the local civil and military authorities, and also, except where they are of minor importance, to the superior Engineer authorities at District, Command, and Army Headquarters. Breaches.

The reopening of the road to traffic, by provision of a diversion or repairs to the breach, must also be similarly reported.

In the case of breaches on roads inside Cantonments, other than through trunk roads, no special immediate report is necessary, except locally.

42. Unduly fast running by heavy mechanical transport ruins the best of macadamized roads by causing corrugations (apart from pot holes), which rapidly increase once they have started. Where there is a tendency of this nature, or when the volume of traffic necessitates it, the speed limits must be enforced with the Speed Limits.

aid of due Military or Police authority. The speed limit for M. T. convoys should not exceed 15 miles per hour in the plains and 10 miles per hour in the hills, apart from special speed limits enforced for all classes of vehicles in dangerous sections. Speed limit notices must be posted at all such places, including bridges which will not stand mechanical transport convoys and heavy vehicles at normal speeds. (See Chapter V, paragraphs 28 to 37 and Chapter IX, paragraph 4.)

MAINTENANCE OF UNMETALLED ROADS.

Unmetalled cart roads.

43. With the exception of metalling, the instructions regarding the maintenance of roads apply fully to unmetalled cart roads.

Where light rough metalling, as described in the general specification for unmetalled cart roads (Chapter III), has been given on soft ground, this should be maintained, and improved similarly to drainage, by the road gangs.

Pack transport roads.

44. In pack transport roads the roadway and drainage, and any structures provided, together with road signs, should be maintained upon similar principles, as applicable.

During the rainy season, small bunds should be made by the road gangs diagonally across the road in hill sections, wherever there is a tendency towards surface scouring.

ROAD IMPROVEMENTS.

Petty improvements.

45. Road improvements in the nature of original works may not be done as repairs, but petty improvements such as cutting back corners, improving side drains, improving camber and superelevation, etc., can and should be done by the repair staff against maintenance estimates, as opportunity offers and funds permit.

Improvements to cart roads to make them fit for M. T.

46. When an existing cart road is to be brought up to M. T. standard, in addition to increasing, widening, and strengthening bridges and culverts, and improving corners, etc., the metalling must be brought up to standard in the thickness as well as in width.

In the case of improvements to an unsoled metalled cart road (except on rocky ground or hard conglomerate) the road must be improved with sufficient metal to bring the total consolidated thickness up to that laid down for metalling *plus* soling for the class of road desired. On rocky ground or hard conglomerate, the total thickness of unsoled metal, including the old metal, need only be brought up to the metalling thickness laid down for the improved standard, where necessary.

CHAPTER XI.

Estimating.

Introductory—Road Construction Estimates—Road Repair Estimates.

INTRODUCTORY.

1. Methods of preparation of estimates, and certain procedure for execution of work are given in this Chapter as a guide; they should be adapted as necessary to suit local circumstances. Introductory.

These instructions should be read in conjunction with the relevant chapters on road construction and maintenance.

ROAD CONSTRUCTION ESTIMATES.

2. A road project for administrative sanction should ordinarily be composed of:— Road Construction Projects.

(i) A report, including a general description of the road and of the work proposed.

(ii) A general specification, appended to the report, (a reference to standard specifications, supported by an explanation of any deviations therefrom, and including general specifications for all road structures, will suffice, where applicable).

(iii) A skeleton map, showing the road alignment on a scale of 2 inches to 1 mile, on which the limits of sections of the road, bridges and causeways, should be marked.

(iv) Abstract estimate of cost.

3. For convenience of estimating and construction, a road, if more than 6 miles long, may be divided into suitable sections, usually of length from 4 to 10 miles. Sections of Road Estimates

The following should constitute separate sections of a road project:—

(i) Estimate for preliminary survey.

(ii) Estimates for large major bridges, including their approaches, defences, and training works, as applicable.

4. The abstract estimate for a road, or for each section thereof, should be complete under the following items, as applicable:— Items of Road Estimates.

I. Road formation.

II. Retaining walls.

III. Training works.

IV. Drains.
V. Scuppers.
VI. Culverts.
VII. Dressing and rolling formation.
VIII. Road parapets.
IX. Metal stacking places.
X. Soling.
XI. Metalling.
XII. Causeways.
XIII. Bridges.
XIV. Road signs (Warning, etc, signs and mile and furlong stones).
XV. Sidings and Parking Places.
XVI. Land compensation.
XVII. Accommodation (temporary accommodation, where necessary, for staff, labour, stores and offices, also rest houses).
XVIII. Water Supply.
XIX. General contingencies, including works charge establishment, flood damage repairs during construction, special tools and plant.
XX. Political charges (where necessary in Frontier Districts).

Large Bridge Projects.

5. A large bridge project should ordinarily be composed of :—

(i) Report
(ii) General specification.
(iii) Calculations for waterway and foundations, and for loads.
(iv) Sketch map showing catchment area (scale about 16 miles to 1 inch will suffice).
(v) Site plan and cross sections (as described in Chapter VI).
(vi) Abstract estimate of cost.

Items of large bridge projects.

6. The estimate for a large girder bridge, for administrative sanction, should be drawn up under the following items, as applicable. In the case of bridges of other types the same general principles should be followed.

I. Training Works.
II. Piers and Abutments.
III. Bridgehead Defences.
IV. Supply of superstructure steelwork.
V. Erection of superstructure steelwork.
VI. Bridge roadway.
VII. Approaches.
VIII. Land compensation.

IX. Accommodation (temporary accommodation for staff, labour, and stores, where necessary).
X. Establishment.
XI. Special tools and plant.
XII. General contingencies, including flood damage repairs during construction.
XIII. Political charges (where necessary in Frontier Districts).

7. Skeleton abstract estimates, for a road section and for a large girder bridge, are appended (Plates LIII and LIV). **Forms of Estimates.**

These illustrate suitable methods for the calculation and explanation of the amounts shown in the abstract estimates. Explanations of rates and costs need not be repeated in successive estimates for sections of a road; they should be explained once in each case and the original explanation or approval should be cited subsequently.

The items should be amplified and modified as necessary to suit varying requirements (*e.g.*, Protection and other political charges, and Serais for labour, are particularly applicable to roads in Frontier Districts, and bridges, etc., of different types require different items and sub-items).

5 per cent. for contingencies should be allowed in the rough detailed estimates supporting the abstract estimate items.

8. In the case of roads in cantonments, a road project should consist of :— **Roads in Cantonments.**

(i) Report.
(ii) General specification.
(iii) Section of cantonment map showing the road.
(iv) Abstract estimate of cost, in suitable concentrated form.

9. The general contingencies provision in a road or bridge estimate should not ordinarily exceed 5 per cent. **General contingencies.**

In the case of Frontier road projects, it may be increased, ordinarily to not more than 10 per cent. Special orders are necessary in such cases.

10. Detailed estimates for the various sub-items of a road or bridge project, for technical sanction, should include, as applicable :— **Detailed Estimates.**

(i) Sheets showing the detailed survey of the road above with the longitudinal section on the same scale below; levels and gradients of the natural ground and proposed road should be given; the sites of scuppers, culverts, causeways, and bridges should be marked and numbered.
(ii) Transverse sections.
(iii) Detailed drawings of all structures proposed.

In estimating for earthwork, the method shown below will be found to be convenient for the calculation of quantities.

Detail of Work	Sectional area in bank	Sectional area in cutting	Mean Section in bank	Mean Section in cutting	Distance between Sections	Quantities Total In bank.	Quantities Total In cutting	Cubic contents to be carried to abstract
1 ROAD FORMATION								
Excavation and Filling Small boulders and gravel								
0 Chainage . .	11 72	2 0	..	..	..	..		.
45 " .	9 23	5 67	10 48	5 73	45	472	240	472
145 " .	61 06	29 50	35 45	19 09	100	3,545	1,909	3,545
245 . .	21 25	24 22	41 45	26 86	100	4,145	2,686	4,145
295 " .	8 6	67 6	14 02	45 91	50	746	2 296	2,296
345 " etc.	60 3	55 1	34 45	61 35	50	1,723	3,068	3,068

Frontier Road Projects.

11. The following remarks apply particularly to Frontier road projects :—

(i) Political charges, which are peculiar to Frontier road projects, particularly in trans-Frontier districts, must be assessed in conjunction with the local Political Agent, through or in conjunction with whom all payments to contractors and tribal labour are, as a rule, to be made, and contracts for work concluded.

(ii) Political charges include :—

Protection (*i e*, cost of tribal guards or badragas).

Land Compensation (*i.e.*, cost of acquisition of land, compensation for crops, irrigation channels, water mills, trees, and graves displaced by the road alignment, etc.).

Royalties on work done by contract (where leviable).

Miscellaneous political charges (*e g.*, M. W. share of expenses of jirgahs at which contracts, etc., are discussed or settled).

(iii) All expenditure on political charges must be certified in each case by the Political Agent and the Assistant Commanding Royal Engineer as being contingent upon the proper execution of the work.

(iv) In cases of special emergency, the waiving of detailed estimates for certain items is sometimes specially sanctioned by the Government of India, *e.g.* :—

All earthwork items (*viz.*, excavation and filling for road formation, sidings, drains, and metal stacking places).

Retaining Walls.

Political Charges.

Temporary serais and godowns for labour and stores.

In such cases, which require special sanction, all measurements of work, and all bills, must be made or checked by a gazetted officer, upon which condition such sanction is contingent.

(v) In special cases, in which, by reason of emergency, estimates for short sections of the road are prepared and sanctioned *seriatim* as the survey proceeds the whole of the section estimates may be incorporated into one omnibus abstract estimate, and an omnibus sanction accorded thereto, in replacement of the separate sanctions to estimates for successive sections directly the estimate for the last section has been prepared. The items of such an omnibus abstract estimate, which does *not* include preliminary survey or large bridge estimates, comprise the main headings I to XX (see para. 4), as applicable, each item comprising the total of the amounts included for the corresponding item in the section abstract estimates; this permits of free co-ordination of work and expenditure throughout the road. Such cases require special sanction

12. Supplementary works (*e.g.* diversions), and repairs, necessary on account of a road, or section of a road, being opened and maintained for traffic before its construction has been completed, should in no case be charged to or included in the project for the construction of the road. Such repairs and works, and the maintenance after completion of a new road or section thereof, or a bridge, must be estimated for and financed separately. **Repairs under traffic.**

13. When any new road has been completed, a correct plan of the road will be sent to the Surveyor-General in India by the Commanding Royal Engineer concerned. **Completion Maps.**

ROAD REPAIR ESTIMATES.

14. Two forms for use in estimating for road maintenance are illustrated in Appendices LV and LVI, *viz.*, Form 1 :—" Estimate of Average cost of Road Maintenance" and Form 2 :—" Abstract Estimate of Remetalling" (detail for sub-head II of Form (1)). These forms, under the system here described, are in use with success in the N.-W. F. Province; the system, with suitable modifications, is suitable for adoption elsewhere. **Repair Estimate Forms.**

15. Form 1 shows in abstract form for each road (or in Cantonments or Civil Stations for suitably classified groups of roads) **Estimate of average expenditure.**

the average annual expenditure required under five parts as follows:—

Part I.—Maintenance (ordinary).
„ II.—Remetalling.
„ III.—Maintenance (special).
„ IV.—Special charges.
„ V.—Arboriculture.

The figures in Form 1 represent averages throughout, and will only vary slightly annually till sufficient experience has been gained: changes in rates and the incidence of traffic will also produce variations from time to time

After the figures in Parts I and III to V have been scrutinized and passed by the sanctioning authority, an allotment in full will be made representing their total *plus* the total of Form 2, and this allotment must be made to cover all normal expenditure.

Form 1 should be submitted to the Commanding Royal Engineer by 15th February.

Abnormal Flood Damages. 16. Abnormal expenditure due to special flood damages will be financed on demand from the central reserve kept by the Commanding Royal Engineer or Chief Engineer. It therefore lies with Assistant Commanding Royal Engineers to submit demands for extra grants due to flood damages when and as they occur. It is incorrect to finance such abnormal repairs from the average grant and make a lump sum demand later on when submitting the statement of changes in grants in the autumn.

Assistant Commanding Royal Engineers will, however, put in hand really urgent flood repairs at once, reporting their action with a rough estimate of funds required, to be followed as soon as possible by an accurate demand based on detailed estimates, on receipt of which the extra allotment will be made.

Arising from these instructions it follows that the average demands first entered in Form 1 should be made on the assumption that no abnormal flood damages will occur. Hence even if the necessity for some abnormal repair is known when Form 1 is prepared, funds for its execution are to be demanded separately and not included under the appropriate sub-head when preparing Form 1.

Abnormal flood damages include such things as the destruction or serious damage of permanent work and unusually heavy slips or causeway clearances

Abstract Estimate of Remetalling. 17. Whilst the *average* expenditure on remetalling is entered in Part II of Form 1, the *actual* amount required for the coming year is carefully estimated and shown in Form 2. Its preparation demands the inspection of every road referred to in Chapter X, paragraph 4, by 15th December.

Form 2 should be submitted to the Commanding Royal Engineer by 5th January.

During January the Commanding Royal Engineer should, as far as possible, traverse every road with the Assistant Commanding Royal Engineer concerned, and pass final orders regarding remetalling so that by the 1st February at latest Assistant Commanding Royal Engineers will be able to place orders for metal collection for those miles which can best be consolidated in April and May. This will enable every sub-district to obtain a much larger output per steam roller than otherwise, which is essential for economic reasons.

Columns 8 to 12 inclusive of Form 2 are only required for record in the Commanding Royal Engineer's office, and must show *actual current rates* for the items concerned, and not schedule rates on which an unknown and variable percentage has to be added to make them comparable with rates in other sub-districts. Details of rates are not required to be submitted with this form. The figure entered in Column 14 must represent the actual funds required to complete the mile concerned, after making allowances for any metal already at site, replacing "reserve" metal, etc. There is no room on the form for any column of remarks, but when explanations are necessary, they should be entered on a separate sheet.

From the above it is clear that the figures entered in Form 2 require the careful scrutiny of the Garrison Engineer and Assistant Commanding Royal Engineer, as they represent actual requirements for remetalling for the year.

Once the grant for this part of Form 1 has been allotted, the Assistant Commanding Royal Engineer has no power to depart from the detailed programme of remetalling approved by his Commanding Royal Engineer, without previous authority, and consequently can only reappropriate *savings* from this to other parts of Form 1.

18. As noted above, Form 2 is due by 5th January, and Form 1 by 15th February. Allotments in full should be made as early as possible after 1st April, for each road or group of roads. Assistant Commanding Royal Engineers have no powers of reappropriation between the allotments so made, and may not exceed any allotment without first obtaining an extra grant (except as provided in paragraph 16 above). Allotments.

19. Accounts will be maintained for each road or group of roads strictly in accordance with the sub-heads entered in Form 1. Accounts.

20. Sanctions given by the Commanding Royal Engineer or Chief Engineer to Forms 1 and 2 are authority for expenditure in advance of allotment within the amounts sanctioned, such ex- Commencement of work.

penditure being incurred against detailed estimates to be sanctioned by the Assistant Commanding Royal Engineer himself, as required.

Additional grants.

21. Applications for extra grants must be accompanied by a clear explanation of the necessity and an abstract of the amount demanded under each part and sub-head of Form 2. As stated in paragraph 16, such demands are always to be made when and as the demand arises, so that the Commanding Royal Engineer or Chief Engineer shall be made acquainted with the cost of serious storm damages immediately after their occurrence.

Road Rollers.

22. When submitting Form 1, Assistant Commanding Royal Engineers should attach to it:—

(i) A statement of road rollers required for the execution of the remetalling programme sanctioned in Form 2.
(ii) An estimate of the cost of ordinary repairs of road rollers.
(iii) An estimate of the cost of moving rollers or of paying their crews when the rollers are idle although fit for work.

As regards (i):—The number of rollers is to be calculated from the total cubical contents of new metalling as sanctioned on Form 1 divided by the average output per roller per working day in cubic feet. The total of roller working days thus obtained is then divided by a figure representing the estimated average output of working days per roller. The calculation is to be reported in detail with an explanation of the data on which the variable factors are used. These statements will enable a proper distribution of all available rollers to be made early in the year by the Commanding Royal Engineer.

As regards (ii):—The estimate for road roller repairs should exclude the cost of running stores and spare parts, the former being debited to consolidation and the latter to Tools and Plant. This estimate is in fact intended to cover the cost of the annual overhaul in the Workshops, which should always be carried out during the hot season, and it must cover the wages of the crews when retained. Owing to the small amount of funds under Tools and Plant, it may often be necessary for the Commanding Royal Engineer to allot funds from his road repair grant to meet the cost of the above estimates, which, however, are properly debitable to the Tools and Plant grant.

As regards (iii):—With proper administration and foresight the expenditure under this head should be small. As it cannot correctly be charged to any particular road, it is not convenient or correct to allow for it under any of the sub-heads of Form 1, so this separate general estimate is necessary.

Repairs to buildings.

23. Attention should be paid to the necessity of charging the repairs and chowkidari of all buildings debitable to roads or com-

munications under the appropriate items in Parts III and IV of Form 1.

24. On a separate sheet attached to Form 1 should be given the following information, as there is no room for it on the form itself :— Miscellaneous details to be attached to estimates.

(i) Names or descriptions of buildings, and their capital costs.

(ii) Necessity for special repairs of above (if any).

(iii) Current wages for road coolies, mates, munshis, malis and chowkidars.

(iv) Explanations of all notable variations from amounts last sanctioned on Form 1 (excluding Form 2).

APPENDIX I.

Summary of ruling dimensions, etc., for metalled roads.

Items.	Class I.	Class II.	Class III.
Land width.			
(a) On irrigated or cultivated land or flat plains.			
Normal	All classes, 60′		
(b) On barren hill sections . . .	Sufficient to allow for catch-water drains and for stacking metal off roadway.		
Gradients.			
(a) Ruling	All classes, 1 in 20		
(b) Maximum	" " 1 in 13.		
Radius of curves.			
Centre line of roadway—			
(a) Normal minimum	All classes, 60′		
(b) Minimum at zigzags in hill sections .	" " 35′		
Super-elevation on curves.			
(a) Maximum, on curves of 35′ radius .	All classes, 1 in 7		
(b) On curves of 50′ radius . . .	" " 1 in 10		
(c) On curves of 100′ radius . . .	" " 1 in 20		
Width of roadway.			
Clear width at formation level, excluding side drains and parapet walls—			
(a) Normal	24′	20′	18′
(b) In straight through or side cuttings in hard rock.	20′	18′	16′
(c) On full embankments exceeding 18″ in height, and in sharply curved and blind through or side cuttings, or sharply curved approaches to bridges and causeways.	27′	24′	22′

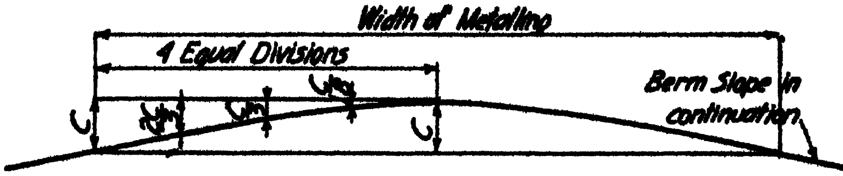
Items.	Class I.	Class II.	Class III.
(d) In tunnels	18′	18′	12′
(e) In metalled dips	..	20′	18′
Outer slopes of embankments.			
(a) Up to 2′ high	All classes, 1 in 4.		
(b) Over 2′ high	Natural slope of soil.		
Height of subgrade.			
(a) In hill sections	All classes, *nil.*		
(b) In plains sections, normal minimum .	,, ,, 6′		
Roadside drains.			
(a) Minimum width	All classes, 2′		
(b) Minimum depth	,, ,, 15′		
Width of soling.			
(a) Normal	1′ more than metalling.		
(b) On high sandy banks or close to high bridge abutments, and in metalled dips.	Clear roadway width.		
(c) On culverts and bridges metalled throughout.	Width of metalling.		
Thickness of soling.			
Before consolidation—			
(a) Normal, using quarried hard rock of large boulders.	All classes, 6″		
(b) Using small boulders or softish material	,, ,, 9″		
(c) On embankments and other soft subgrades and in metalled dips.	,, ,, 9″ to 12″		
Width of metalling.			
(a) Normal	16′	12′	9′
(b) On curves of 100′ radius or less having a subtended angle of 45 or more.	20′	16′	12′
(c) On bridges and culverts . . .	Full width between wheel guards.		
Thickness of metalling.			
Layers before consolidation	2 × 4½″	1 × 6″	1 × 4½″

Items.	Class I.	Class II.	Class III.
Camber of road surface.			
Crown slopes—			
(a) Normal, in straights	All classes, 1 in 40.		
(b) On curves	No camber, super-elevation cross slope.		
Width of berms.			
(a) Clear width of roadway on either side outside metalling excluding road parapets and side drains, normal.	4′	4′	4½′
(b) Do. range	2′—5½′	3′—6′	3½′—6½′
(c) Overall width	4′—8½′	5′—9′	5½′—9½′
Slope of berms.			
(a) Normal in straights	All classes, 1 in 40.		
(b) On curves	Super-elevation cross slope.		
Width of culverts.			
(Total length between abutments not exceeding 12′).			
Clear width between wheel guards—			
(a) Normal	24′	20′	18′
(b) Minimum	20′	18′	16′
(c) Maximum	27′	24′	22′
Width of Scuppers.			
(Paved dips not exceeding 20′ span)	All classes, formation width.		
Width of bridges.			
Minor bridges :—total length between abutments not exceeding 100′.			
Major bridges :—total length between abutments exceeding 100′.			
Clear width between wheel guards—			
(a) Normal	18′	18′	16′
(b) Small bridges on sharp curves	22′	22′	14′
Strength of bridges and culverts	Calculated to take 12 ton steam roller with 25% impact.		

Items.	Class I.	Class II.	Class III.
Width of causeways.			
Overall width	18′	18′	12′
Width of overflow bridges.			
Overall width	18′	18′	12′
Height of parapets and hand rails.			
Above road surface, normal—			
(a) On roadside parapet walling . . .	All classes, 2′		
(b) On culverts	,, ,, 1½′		
(c) On bridges	,, ,, 2′		
Head-room in tunnels and over bridges.			
(a) Over centre 8′ of roadway . . .	All classes, 13′		
(b) Over side of roadway, minimum . .	,, ,, 8′		
Sidings.			
(When specially ordered).			
Normal dimensions—			
(a) Maximum length	All classes, 66′		
(b) Maximum width	,, 16′		

PLATE II.

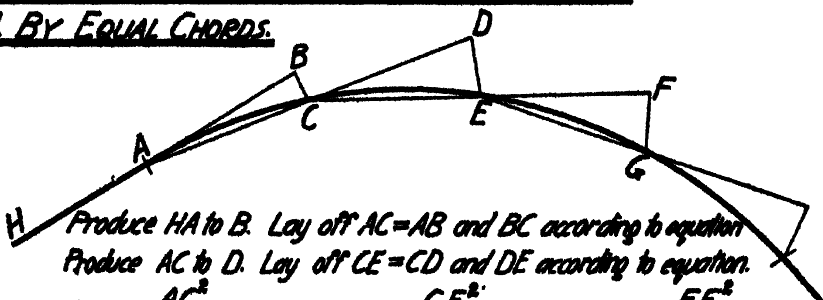
CAMBER CURVE FOR ROAD SURFACE.



C is the camber given by the crown slopes specified

THREE METHODS OF SETTING OUT CURVES.

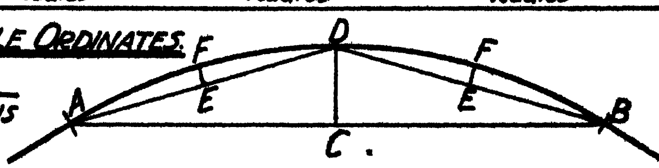
1. BY EQUAL CHORDS.



Produce HA to B. Lay off AC = AB and BC according to equation.
Produce AC to D. Lay off CE = CD and DE according to equation.

$$BC = \frac{AC^2}{2 \times Radius} \qquad DE = \frac{CE^2}{Radius} \qquad FG = \frac{EF^2}{Radius}$$

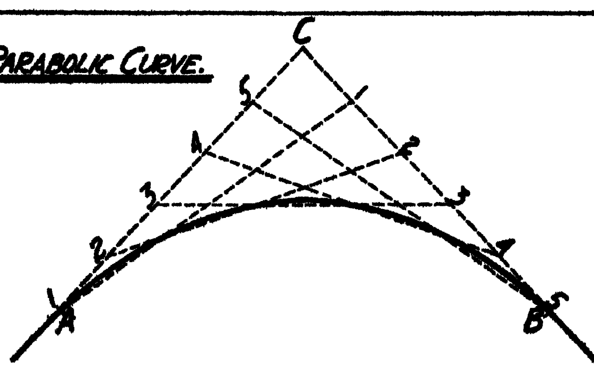
2. BY MIDDLE ORDINATES.



$$DC = \frac{AB^2}{8 \times Radius}$$

$$EF = \frac{DC}{4}$$

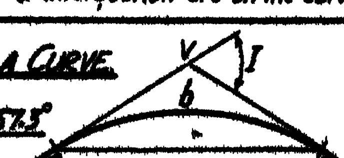
3. BY A PARABOLIC CURVE.



Divide AC and CB into a number of equal parts and number as shown.
Join 1-1, 2-2, 3-3 etc. The interior points of intersection are on the curve.

GENERAL EQUATIONS TO FIND THE RADIUS OF A CURVE.

$$R = \frac{ac}{2 \sin\left(\frac{I}{2}\right)} = vc \cot\left(\frac{I}{2}\right) = \frac{arc\ abc \times 57.3^\circ}{I}$$



APPENDIX III.

Summary of ruling dimensions, etc., for unmetalled roads.

Items.	Class IV.	Class V.	Class VI.
Land width.			
(a) On irrigated or cultivatable land, or flat plains, normal.	60′	16′	14′
(b) On barren hill sections	Sufficient to allow for catch-water drains.		
Gradients.			
(a) Ruling	1 in 20	1 in 13	1 in 8
(b) Maximum	1 in 13	1 in 8	1 in 6
Radius of curves			
Centre of roadway—			
(a) Normal minimum	60′	20′	10′
(b) At zigzags in hill sections	35′	10′	6′
Width of roadway.			
Clear width at formation level, excluding side drains and parapet walls—			
(a) Normal	12′	10′	8′
(b) On full embankments exceeding 18″ in height and at blind corners.	15′	12′	10′
Outer slopes of embankments.			
Normal	Natural slope of soil.		
Height of subgrade.			
(a) In hill sections	*Nil*	*Nil*	
(b) In plains sections, normal minimum .	6″		
Roadside drains.			
(a) Minimum width	All classes, 2′		
(b) Minimum depth	„ „ 15″		

Items.	Class IV.	Class V.	Class VI.
Camber of road surface.			
(a) Crown slopes, normal	1 in 30	1 in 20-30	1 in 20-30.
(b) In hill sections	1 in 30	1 in 20-30, or cross slope 1 in 20-30.	
Width of culverts.			
(Total length between abutments not exceeding 12′).			
Clear width between wheel guards, normal .	12′	(where necessary). 10′	8′
Width of scuppers.			
Paved dips not exceeding 20′ span . .	All classes, formation width. (where necessary).		
Width of bridges.			
(Minor bridges, total length between abutments not exceeding 100′.			
Major bridges, total length between abutments exceeding 100′).			
Clear width between wheel guards, normal .	10′	(where necessary). 8′	8′
Strength of bridges & culverts.			
Normal	Calculated to take 12 ton steam roller with 25% impact.	(where necessary) Calculated to take Infantry crowded at a check.	
Width of causeways.			
Overall width	12′	..	..
Width of overflow bridges.			
Overall width	12′	..	..
Height of parapets and hand rails.			
Above road surface, normal—			
(a) In roadside parapet walling . . .	All classes, 2′		
(b) On culverts	„ „ 1½′		
(c) On bridges	„ „ 2′		

PLATE IV.

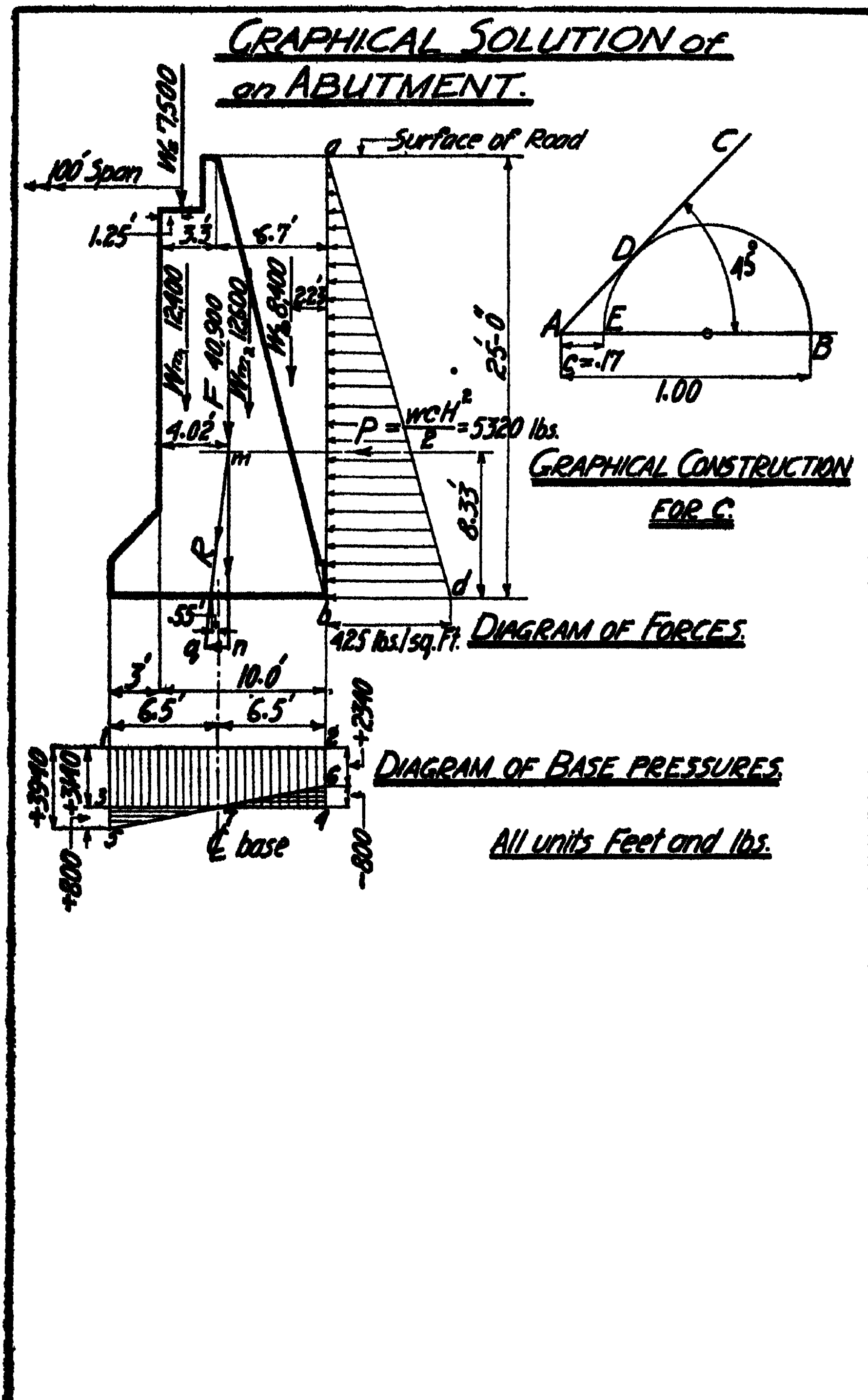
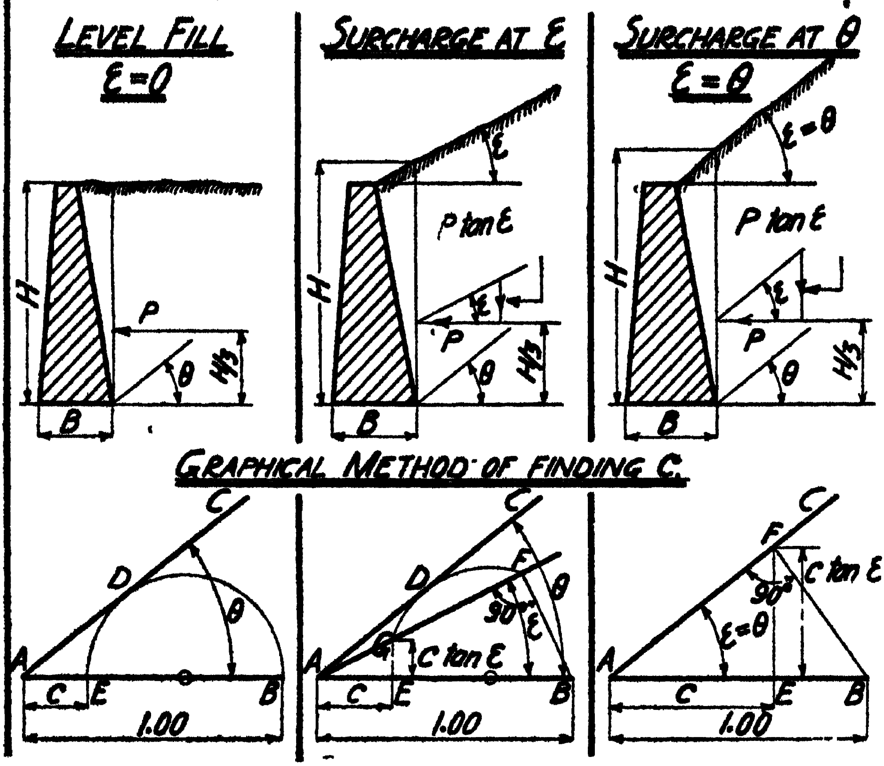

GRAPHICAL SOLUTION of an ABUTMENT.
100' Span
W3 7,500
Surface of Road
a
1.25'
3.3'
6.7'
Wm1 12,400
F 40,900
Wm2 12,600
W2 8,400
2.2'
25'-0"
P = wcH²/2 = 5320 lbs.
4.02'
m
8.33'
R
d
b
.55'
q
n
425 lbs/sq. Ft.
DIAGRAM OF FORCES.
3'
10.0'
6.5'
6.5'
+2340
+3940
+3140
+800
-800
℄ base
DIAGRAM OF BASE PRESSURES.
All units Feet and lbs.
C
D
45°
A
E
B
C=.17
1.00
GRAPHICAL CONSTRUCTION FOR C.

PLATE V.

PRESSURES ON RETAINING WALLS.



Draw to any Scale AB=1·00 and AC at angle θ to AB.

With centre on AB draw arc BDE touching AC at D. AE=c to same scale as AB.	Draw AF at angle ε. BF perpendicular to AF. With centre on AB draw arc FDG touching AC at D. Draw GE perpendicular to AB. AE=c GE=c tan ε	Draw BF perpendicular to AF and FE perpendicular to AB. AE=c EF=c tan ε.

EQUATIONS FOR C.

$c=\dfrac{1-\sin\theta}{1+\sin\theta}$	$c=\cos^2\varepsilon\,\dfrac{\cos\varepsilon-\sqrt{\cos^2\varepsilon-\cos^2\theta}}{\cos\varepsilon+\sqrt{\cos^2\varepsilon-\cos^2\theta}}$	$c=\cos^2\theta$
$P=\dfrac{cwH^2}{2}$	$P=\dfrac{cwH^2}{2}$	$P=\dfrac{cwH^2}{2}$

PLATE VI.

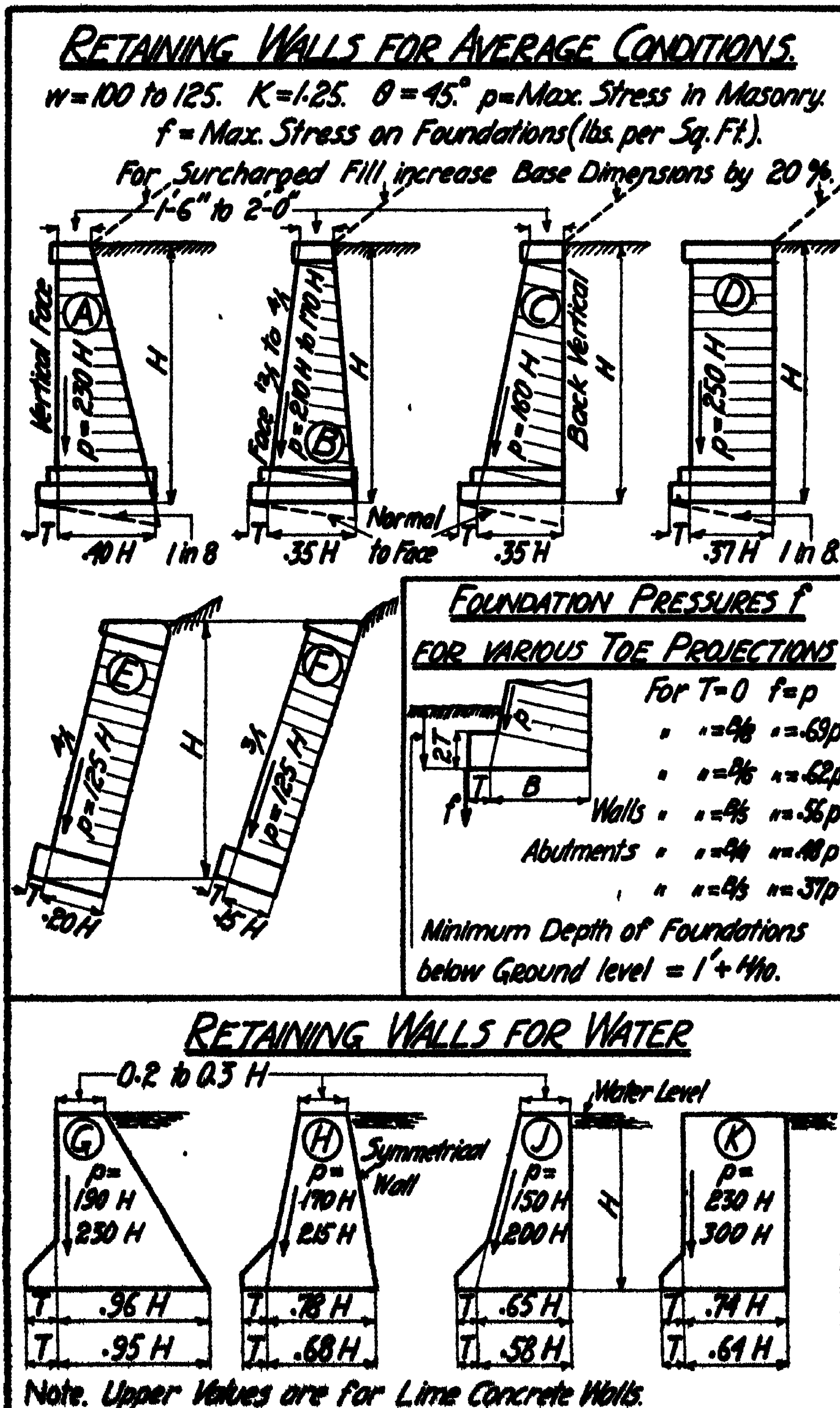

RETAINING WALLS FOR AVERAGE CONDITIONS.
w=100 to 125. K=1·25. θ=45° p=Max. Stress in Masonry.
f=Max. Stress on Foundations (lbs. per Sq. Ft.).
For Surcharged Fill increase Base Dimensions by 20%.
1'-6" to 2'-0"
Vertical Face
A
p=230 H
H
·40 H
1 in 8
Face 1/12 to 1/4
p=210 H to 170 H
B
·35 H
Normal to Face
C
p=160 H
Back Vertical
·35 H
D
p=250 H
·37 H
1 in 8
T
E
1/4
p=125 H
·20 H
F
3/8
p=125 H
·45 H
FOUNDATION PRESSURES f FOR VARIOUS TOE PROJECTIONS
For T=0 f=p
" "=B/8 "=.69p
" "=B/6 "=.62p
Walls " "=B/5 "=.56p
Abutments " "=B/4 "=.48p
" "=B/3 "=.37p
2T
P
T
B
f
Minimum Depth of Foundations below Ground level = 1'+H/10.
RETAINING WALLS FOR WATER
0·2 to 0·3 H
Water Level
G
p= 190 H 250 H
·96 H
·95 H
H
p= 170 H 215 H
Symmetrical Wall
·78 H
·68 H
J
p= 150 H 200 H
·65 H
·58 H
K
p= 230 H 300 H
·74 H
·64 H
Note. Upper Values are for Lime Concrete Walls.

TABLE VII.

Retaining Walls.

Values of $\frac{\text{Base}}{\text{Height}}$ for a top thickness of $\frac{\text{Height}}{10}$ (except Rectangular walls).

May be used for top thicknesses of H/3 to H/30 without appreciable error
Base pressures are for W=100 lbs. per cu. Ft. For heavier Fills increase Base pressures in proportion.

Equation No.	Type of wall.	Level Fill B/H Equals.	Equation No.	Type of wall.	Level Fill B/H Equals.
1	Face Vertical .	$\sqrt{c}$	6	Face 4/1 .	$\sqrt{c+\cdot07}-\frac{K}{8}$
2	Face 24/1 . .	$\sqrt{c}-\frac{K}{48}$	7	Vertical Back .	$\sqrt{\frac{c}{K}+\cdot01}-\cdot05$
3	Face 12/1 . .	$\sqrt{c}-\frac{K}{24}$	8	Rectangular Section	$\sqrt{\frac{c}{K}}$
4	Face 8/1 .	$\sqrt{c+\cdot01}-\frac{K}{16}$	9	Symmetrical Section	$\sqrt{\frac{3c}{\sqrt{K}}}$
5	Face 6/1 . .	$\sqrt{c+\cdot03}-\frac{K}{12}$	10	Face n/1 Back Vertical	$\sqrt{\frac{c}{K}+\frac{1\cdot25}{n^2}}-\frac{1}{2n}$

EQUATION No.				1	3	4	5	6	7	8	9
	Angle of Internal Friction			Values of B/H.							
				Face Batter					Back Vertical	Rectangular Section	Symmetrical Section.
	Slope.	Degree.	c	Vertical	12/1	8/1	6/1	4/1			
K=1·5	¾ to 1	53°	·11	33	27	25	·25	24	·24	27	·27
	1 to 1	45°	·17	·41	35	33	32	·30	·30	34	·33
	1⅓ to 1	37°	·25	·50	44	41	·40	·38	·37	·41	·40
	1½ to 1	34°	28	·54	·47	·45	·44	·41	·40	·44	·43
	2 to 1	27°	·38	·62	·56	·53	52	·48	·46	·50	·50
p=Maximum Toe pressure lbs. per sq ft.				260 H	243 H	232 H	221 H	200 H	190 H	300 H	225 H
Factor of Safety—overturning .				3 60	2 90	2 65	2·50	2·22	2 03	3 00	2·49
K=1·25	¾ to 1	53°	·11	·33	·28	27	·27	27	·26	·30	·28
	1 to 1	45°	·17	·41	·36	35	34	·34	·32	·37	·35
	1⅓ to 1	37°	·25	·50	·43	43	42	·41	·41	·45	·42
	1½ to 1	34°	28	·54	·48	·47	·46	·44	·44	·48	·45
	2 to 1	27°	·38	62	·57	·55	·54	·52	·51	·55	·52
p=Maximum Toe pressure lbs. per sq ft.				230 H	211 H	200 H	190 H	170 H	156 H	250 H	192 H
Factor of Safety—overturning .				3 30	2 70	2 52	2 40	2·14	2 04	3·00	2·39
K=1·00	¾ to 1	53°	·11	·33	29	29	·29	·30	·30	·33	·30
	1 to 1	45°	·17	·41	·37	·36	·37	·37	·38	·41	·37
	1⅓ to 1	37°	·25	·50	·46	·45	·45	·44	·46	·50	·45
	1½ to 1	34°	·28	·54	·49	·48	·48	·47	·49	·54	·48
	2 to 1	27°	·38	·62	·58	·56	·56	·55	·57	·62	·55
p=Maximum Toe pressure lbs per sq. ft.				200 H	182 H	172 H	163 H	143 H	122 H	200 H	161 H
Factor of Safety—overturning .				3 00	2 50	2 35	2 22	2·07	2·02	3·00	2·21

Note.—In all cases the position of the Resultant is Sliding tan $\mu = \frac{cwH^2}{2V+pB}$

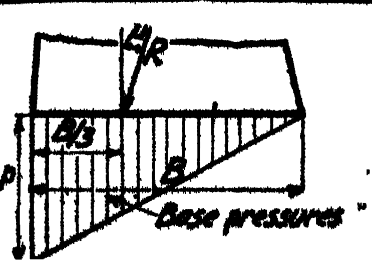


TABLE VIII.

Waterway areas required for catchment areas.

Based on the Dun Drainage Table.

Catchment Area in sq. Miles.	Area of Waterway in sq. ft. In Hills 120%	Area of Waterway in sq. ft. In Hills 100%	Area of Waterway in sq. ft. In Plains 80%	Area of Waterway in sq. ft. In Plains 50%	Catchment Area in sq. Miles.	Area of Waterway in sq. ft. In Hills 120%	Area of Waterway in sq. ft. In Hills 100%	Area of Waterway in sq. ft. In Plains 80%	Area of Waterway in sq. ft. In Plains 50%
·01	2·4	2·0	1·6	1·0	12	888	740	592	370
·02	4·8	4·0	3·2	2·0	14	966	805	644	403
·04	9·0	7·5	6·0	3·8	16	1038	865	692	433
·06	12·6	10·5	8·4	5·3	18	1104	920	736	460
·08	16·2	13·5	10·8	6·8	20	1164	970	776	485
·10	19	16	13	8	25	1296	1080	864	540
·20	38	32	26	16	30	1416	1180	944	590
·30	53	44	35	22	35	1528	1273	1018	637
·40	67	56	45	28	40	1620	1350	1080	675
·50	79	66	53	33	50	1812	1510	1208	755
·60	89	74	59	37	60	1980	1650	1320	825
·80	106	88	70	44	70	2136	1780	1424	890
1·0	120	100	80	50	80	2280	1900	1520	950
1·2	144	120	96	60	90	2418	2015	1612	1008
1·4	168	140	112	70	100	2544	2120	1696	1060
1·6	192	160	128	80	120	2778	2315	1852	1156
1·8	216	180	144	90	140	3000	2500	2000	1250
2·0	240	200	160	100	160	3198	2665	2132	1333
2·5	300	250	200	125	180	3384	2820	2256	1410
3·0	360	300	240	150	200	3564	2970	2376	1485
3·5	419	349	279	175	250	3970	3308	2646	1654
4·0	466	388	310	194	300	4338	3615	2892	1808
4·5	509	424	339	212	400	4998	4165	3332	2083
5	546	455	364	228	500	5532	4610	3688	2305
6	611	509	407	255	600	6036	5030	4024	2515
7	667	556	445	278	700	6504	5420	4336	2710
8	721	601	481	301	800	6960	5800	4640	2900
9	769	641	513	321	900	7296	6080	4864	3040
10	815	679	543	340	1000	7656	6380	5104	3190

PLATE IX.

SCOUR and AFFLUX NOTATION.

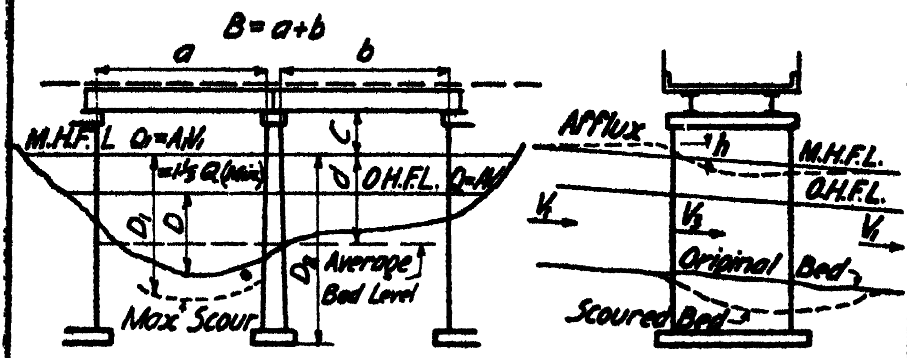


TABLE X.

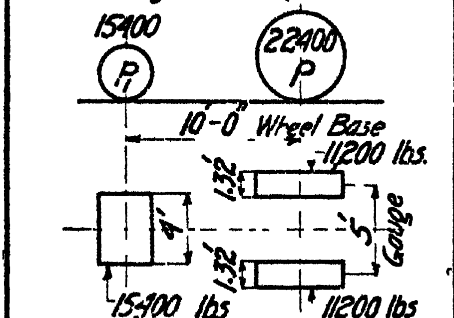
Table of powers for use in Equations.

X	$X^{\frac{2}{3}}$	$X^{1.64}$	$X^{.64}$	$X^{\frac{1}{2}}$	X	$X^{\frac{1}{2}}$
1	1·0	1·0	1·0	1·0	42	6·48
2	1·59	3·12	1·56	1·41	44	6·63
3	2·08	6·06	2·02	1·73	46	6·78
4	2·52	9·69	2·42	2·00	48	6·93
5	2·92	14·0	2·80	2·24	50	7·07
6	3·30	18·9	3·15	2·45	52	7·21
7	3·66	24·3	3·47	2·65	54	7·35
8	4·00	30·3	3·79	2·83	56	7·48
9	4·33	36·8	4·09	3·00	58	7·62
10	4·64	43·7	4·37	3·16	60	7·75
11	4·94	51·1	4·64	3·32	62	7·87
12	5·24	58·9	4·91	3·46	64	8·00
13	5·53	67·2	5·17	3·61	66	8·12
14	5·81	75·9	5·42	3·74	68	8·25
15	6·08	85·2	5·68	3·87	70	8·37
16	6·35	94·7	5·91	4·00	72	8·49
17	6·61	105·0	6·19	4·12	74	8·60
18	6·87	115	6·40	4·24	76	8·72
19	7·12	125	6·58	4·36	78	8·83
20	7·37	136	6·80	4·47	80	8·94
21	7·61	148	7·05	4·58	82	9·06
22	7·85	159	7·24	4·69	84	9·17
23	8·09	171	7·44	4·80	86	9·27
24	8·32	184	7·68	4·90	88	9·38
25	8·55	196	7·85	5·00	90	9·49
26	8·77	209	8·04	5·10	92	9·59
27	9·00	223	8·26	5·20	94	9·70
28	9·22	237	8·46	5·29	96	9·80
29	9·44	252	8·68	5·39	98	9·90
30	9·65	265	8·81	5·48	100	10·00
32	10·08	295	9·22	5·66	..	..
34	10·49	326	9·60	5·83	..	..
36	10·90	358	9·95	6·00	..	..
38	11·30	388	10·20	6·16	..	..
40	11·70	425	10·62	6·32	..	..

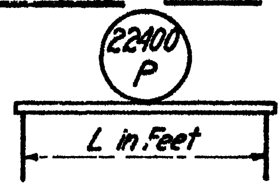
PLATE XI.

LOADS AND MOMENTS IN SLABS, STRINGERS AND FLOORBEAMS

12 TON ROAD ROLLER
Including 25% Impact

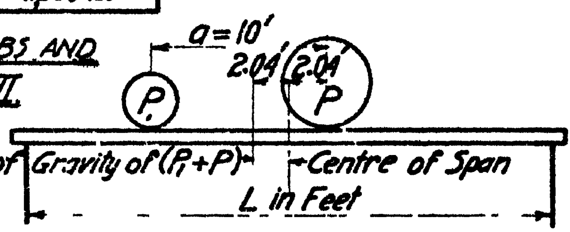


LONGITUDINAL SLABS AND STRINGERS. Case I.



$$\text{Moment} = \frac{PL}{4}\ (\text{Ft. lbs.})$$

LONGITUDINAL SLABS AND STRINGERS. Case II.



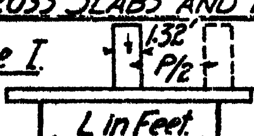
The Moment is greater for Case II for Spans greater than

$$L = \frac{a \text{ i.e. } 10'}{r+1-\sqrt{r^2+r}} = 17.7 \text{ Ft., where } r = \frac{P}{P_1} = \frac{22400}{15400} = 1.455 \text{ and}$$

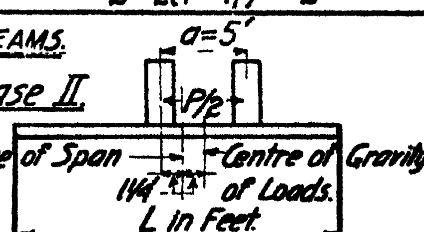
$$\text{Max. Moment at } P = [P+P_1]\frac{x^2}{L} \text{ and } x = \frac{L}{2} - \frac{P_1 a}{2(P+P_1)} = \frac{L}{2} - 2.04'$$

CROSS SLABS AND FLOOR BEAMS.

Case I.



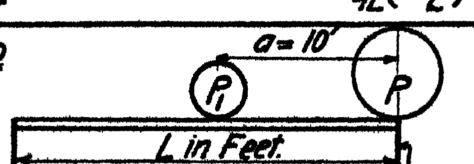
Case II.



Case I. $\text{Moment} = \frac{P}{4}\left(\frac{L}{2} - \frac{1.32}{4}\right)$

Case II. (see above) $r = 1$. The Moment is greater for Case II for spans greater than $L = \frac{a \text{ i.e. } 5'}{r+1-\sqrt{r^2+r}} = 8.53'$ and $\text{Max. Moment at } P/2 = \frac{P}{4L}\left(L - \frac{a}{2}\right)^2$

MAXIMUM REACTION OR LOAD ON A PIER OR FLOOR BEAM.
For every 10' Width of Road



Case I. Spans under 10'. Max. Reaction = P

Case II. Spans over 10'. Max Reaction = $P + P_1\left(\frac{L-a}{\ldots}\right)$

TABLE XII.

Moments in longitudinal Slabs, Stringers and Girders for 12-Ton Road Roller *plus* 25 per cent. Impact. Spans 1′ to 50′.

Span L c to c Bearings.	Moments for Road Roller.	Moment on 5′ width of Slab for Stringers.	Steel Stringers. Moment per foot width for Stringers.	Steel Stringers. Modulus per ft. Width. No Wearing Surface.	Steel Stringers. Modulus per ft. Width. Wearing Surface 70 lbs. per sq. ft.	Effective width of Slab ⅔ L+c.	Moment per foot width for Slabs	Total Thickness of Reinforced Concrete Slabs. No Wearing Surface. Simple	Total Thickness of Reinforced Concrete Slabs. No Wearing Surface. Continuous.	Total Thickness of Reinforced Concrete Slabs. Wearing Surface 70 lbs. per sq. ft. Simple.	Total Thickness of Reinforced Concrete Slabs. Wearing Surface 70 lbs. per sq. ft. Continuous.
Ft.	Ft. lbs.	Ft. lbs.	Ft. lbs.	Z	Z	Ft.	Ft. lbs.	Ins.	Ins.	Ins.	Ins.
1	2	3	4	5	6	7	8	9	10	11	12
1	5,600	2,800	560	·406	·416	1·99	1,409	5	5	5	5
2	11,200	5,600	1,120	·827	·865	2·65	2,110	5¾	5	6	5
3	16,800	8,400	1,680	1·26	1·32	3·32	2,524	6¼	5½	6½	5½
4	22,400	11,200	2,240	1·71	1·81	3·99	2,809	6¾	5¾	7	5¾
5	28,000	14,000	2,800	2·17	2·34	4·65	3,010	7¼	6	7½	6¼
6	33,600	16,800	3,360	2·64	2·88	5·0	3,360	7¾	6½	8	6¾
7	39,200	19,600	3,920	3·20	3·46	5·0	3,920	8¼	6¾	8½	7¼
8	44,800	22,400	4,480	3·63	4·06	5·0	4,480	8¾	7¼	9¼	7¾
9	50,400	25,200	5,040	4·14	4·69	5·0	5,040	9½	7¾	9¾	8¼
10	56,000	28,000	5,600	4·67	5·34	5·0	5,600	10	8¼	10¼	8¾
11	61,600	30,800	6,160	5·21	6·02	5·0	6,160	10¾	8¾	11¼	9¼
12	67,200	33,600	6,720	5·76	6·73	5·0	6,720	11¼	9¼	12	9¾
13	72,800	36,400	7,280	6·33	7·46	5·0	7,280	12	9¾	12¾	10¼
14	78,400	39,200	7,840	6·91	8·23	5·0	7,840	12¾	10¼	13½	11¼
15	84,000	42,000	8,400	7·51	9·01	5·0	8,400	13¼	11	14¼	12
16	89,600	44,800	8,960	8·11	9·83	5·0	8,960	14	11¾	15	12¼
17	95,200	47,600	9,520	8·74	10·67	5·0	9,520	14¾	12¼	16	13
18	101,727	50,864	10,173	9·44	11·61	5·0	10,173	15½	12¾	16¾	13¾
19	110,717	55,359	11,072	10·33	12·74	5·0	11,072	16¼	13¼	17½	14¼
20	119,743	59,872	11,074	11·23	13·01	5·0	11,074	17¼	14	18½	15¼
22	137,938	68,969	13,794	13·00	16·34						
24	156,242	78,121	15,624	15·02	18·87						
26	174,631	87,316	17,463	17·00	21·53						
28	193,106	96,553	19,311	19·04	24·29						
30	211,632	105,816	21,163	21·14	27·17						
32	230,204	115,102	23,020	23·30	30·16						
34	248,815	124,408	24,882	25·51	33·26						
36	267,458	133,729	26,746	27·78	36·40						
38	286,128	143,064	28,613	30·11	39·78						
40	304,821	152,411	30,482	32·49	43·20						
42	323,533	161,767	32,353	34·92	46·79						
44	342,263	171,132	34,226	37·41	50·38						
46	361,008	180,504	36,101	39·98	54·13						
48	379,765	189,883	37,977	42·50	57·08						
50	398,531	199,267	39,853	45·21	61·95						

TABLE XIII.

Moments in Cross Slabs and Reactions on Piers and Floor Beams for 12-Ton Road Roller *plus* 25 per cent. Impact.

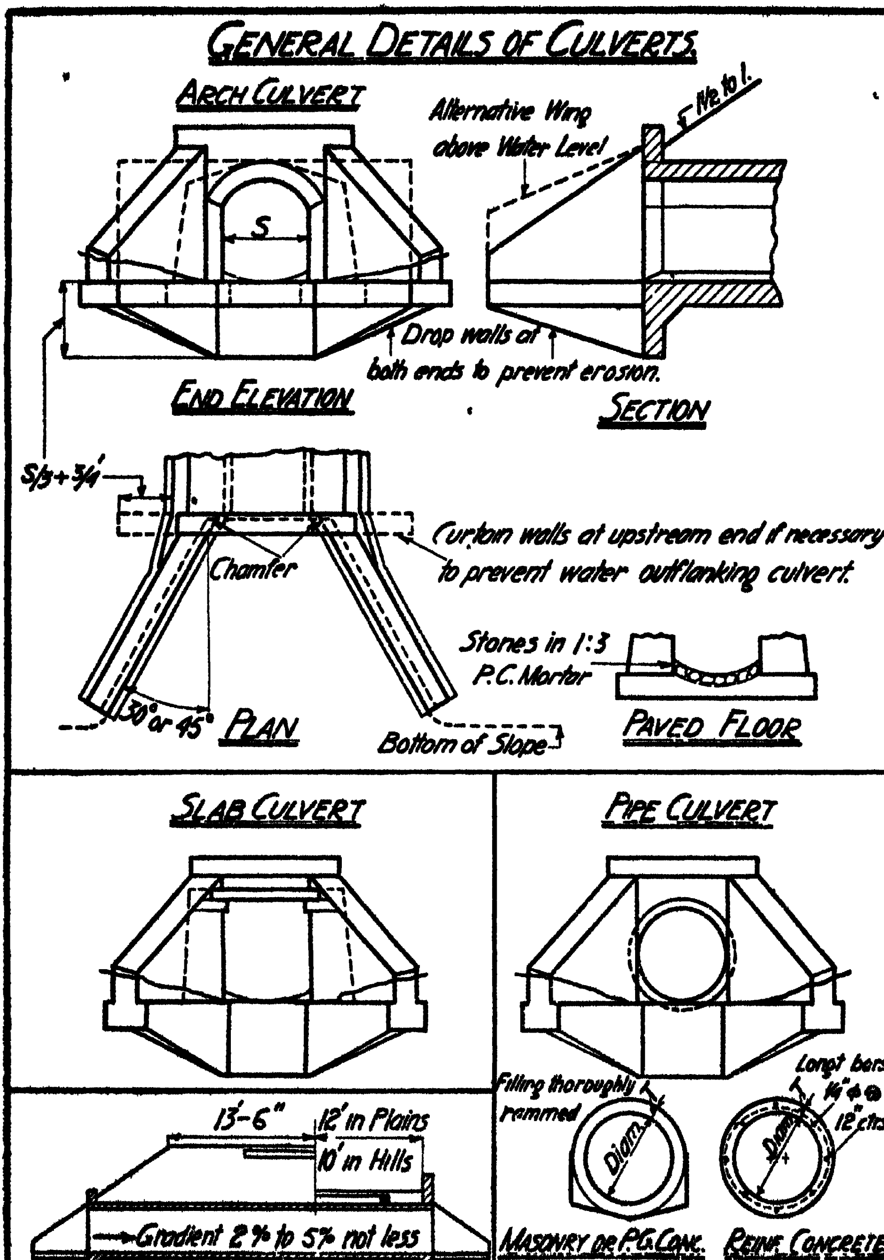
Span L c to c. bearings	Moment for Road Roller Rear wheels only	Effective width of slab ⅔(L+c)	Moment per ft width for slabs	Total Thickness of Reinforced Concrete Slabs: No wearing surface, Simple	No wearing surface, Continuous	Wearing surface 70 lbs per sq ft, Simple	Wearing surface 70 lbs per sq ft, Continuous	Spacing of Floor-beams or Piers	Reaction of Road Roller on Floor beam or Pier	Spacing of Floor beams	Load per ft. of floor-beam span
Ft	Ft lbs	Ft	Ft lbs	Ins	Ins	Ins	Ins	Ft	lbs	Ft.	lbs
1	2	3	4	5	6	7	8	9	10	11	12
1	1,060	1 33	795	5	5	6	5	1	22,400	1	448
2	3 752	2 21	1,698	5½	5	5½	5	2	22 400	2	896
3	6,552	2 88	2 275	6	5	6½	5½	3	22,400	3	1,344
4	9,352	3 55	2,630	6¾	5¼	6¾	5½	4	22,400	4	1,7[illegible]
5	12,152	4 21	2,886	7	5¾	7¼	6	5 to 10	22,400	5 to 10	2,240
6	14,952	4 88	3,064	7½	6	7½	6½	11	[illegible]	11	2 380
7	17,752	5 55	3,199	7¾	6½	8	6¾	12	24,367	12	2,467
	20,552	6 21	3,310	8	6¾	8½	7¼	13	25,954	13	2,545
9	26,289	6 88	3,621	8½	7¼	9	7½	14	26,800	14	2,650
10	31,500	7 55	4,172	9	7½	9½	8	15	27,563	15	2,753
11	36,782	8 21	4,460	9½	8	10½	8¾	16	28,175	16	2,818
12	42,117	8 88	4 748	10	8½	11	9	17	28,741	17	2,874
13	47,492	9 55	4,973	10¼	8¾	11½	9½	18	29,244	18	2,924
14	52,900	10 0	5,290	11	9	12½	10¼	19	29,695	19	2 970
15	58,333	10 0	5,833	11¾	9¾	13	10¾	20	30,100	20	3,010
16	63,788	10·0	6 379	12½	10¼	13½	11½	25	31,640		
17	69,259	10·0	6,926	13	10¾	14¼	12	30	32,667		
18	74,747	10·0	7,475	13¾	11½	15½	12¾	35	33,400		
19	80,242	10·0	8,024	14½	12	16½	13½	40	33,950		
20	85,750	10·0	8,575	15½	11¾	17½	14½	50	34,720		

TABLE XIV.

pacing of Bars in Slabs.

Total Thickness of Slab.	Depth below centre of steel.	Steel Area per ft. width.	Spacing in inches of Round Bars for Diameters of					Total Thickness of Slab.	Depth below centre of steel.	Steel Area per ft. width.	Spacing in inches of Round Bars for Diameters of				
Ins.	Ins.	Sq. Ins.	3/8″	1/2″	5/8″	3/4″	7/8″	Ins.	Ins.	Sq. Ins.	5/8″	3/4″	7/8″	1″	1 1/8″
5	1	·322	4	7¼				11	1½	·764	4¾	6¾	9½		
5½	1	·362	3½	6½				12	1½	·844	4¼	6¼	8½		
6	1	·402	3¼	5¾	9			13	1½	·925	4	5¾	7¾	10	
6½	1¼	·423	3	5½	8½			14	1½	1·005	3¾	5¼	7	9¼	
7	1¼	·463	2¾	5	7¾			15	1¾	1·06	3½	5	6¾	8¾	
7½	1¼	·503	2½	4½	7¼	10½		16	1¾	1·15		4½	6¼	8¼	10¼
8	1¼	·543	2¼	4¼	6¾	9¾		17	1¾	1·23		4¼	5¾	7½	9½
9	1¼	·623	2	3¾	5¾	8½	11½	18	1¾	1·31		4	5½	7¼	9
10	1¼	·704		3¼	5	7½	10	19	1¾	1·39			5	6¾	8½

PLATE XV.



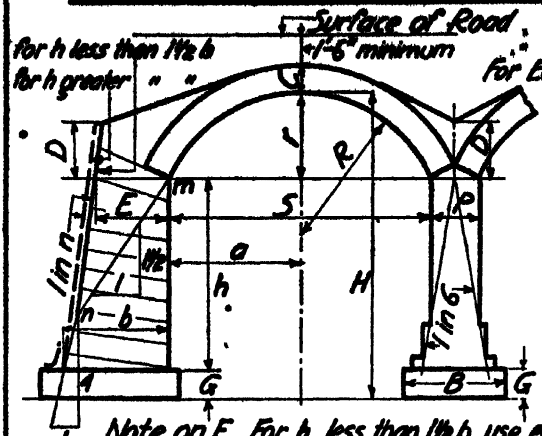
SECTION IN BANK | SECTION NO BANK

For details see the following Plates:
Arch rings Pl XVI
Slabs Pl XXII Troughing Pl XXI Piers Pl XVIII
Abutments and Wing walls Pl XVII

Diam	Thickness of Shell T			Hoop Bars.
	Masonry	P.C. Conc.	Reinf. Conc.	
2'	9"	6"	3"	¼"φ @ 4½"
3'	9"	7"	3½"	¼"φ @ 3½"
4'	10"	8"	4"	5/16"φ @ 4½"
5'	[illegible]	[illegible]	[illegible]	[illegible]

PLATE XVI.

ARCH BRIDGES—EQUATIONS AND DIMENSIONS



For Economy Span $S = \cdot 8H$ to $5H$ (1).

$$R = \frac{a^2 + r^2}{2r} \quad (2).$$

$$C = \frac{\sqrt{R+a}}{4} + 0.2 \quad (3).$$

$$E = \frac{R}{5} + \frac{r}{10} + 2' \text{ (see note) } (4)$$

$$n = \frac{24r}{S} \quad (5) \qquad D = \frac{r+C}{2} \quad (6)$$

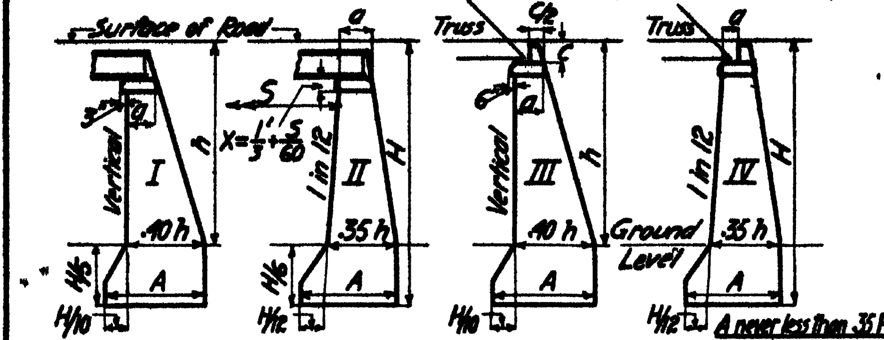
All units Feet

Note on E. For h less than 1½ b use equation (4) for E. When h exceeds 1½ b from m draw mn at 1½ to 1 intersecting batter 1 in n from E in n. From n draw nj at 1 in 4 intersecting base at j, from j draw new back batter of 1 in n

Spans in Feet	2	3	4	5	6	10
For r/S = ½ to ⅙.						
1. For 1 : 2 : 4 P.C. Concrete. Equation (3) . . . . C=	8"	9"	10"	11"	12"	14"
2. For H.D.V. Stone in Lime . C—	9"	9"	10"	12'	12"	15"
3. For Hard Burnt Brick in Lime . C =	9"	9'	13½"	13½"	13½"	18"
4. For Best Lime Concrete . . C—	12"	12"	14"	14"	16"	20"
5. E for r/S = ½ to ⅙ (h less than 1½ b)	1' 6"	1' 6"	2' 0"	2' 4"	2' 8"	3' 8"
6. P for P.C. Concrete or H.D. Stone in Lime.	1 0"	1' 4"	1' 7"	1' 11"	2' 1"	2' 6"
7 P for Brick in Lime Mortar . .	1' 1½"	1' 6"	1' 10½"	2' 3"	2' 3"	2' 7½"
8. G for 1 : 3 : 6 P.C. Concrete . .	1' 0"	1' 0"	1' 0"	1' 3"	1' 3"	1' 6"
9. G. for best Lime Concrete . .	1' 6"	1' 6"	1' 6"	1' 9'	1' 9"	2' 0'
10. Load for Pier Foundations (see note below) . . Tons	3·5	4·0	4·5	5 0	5 5	7·5
11. Load for Abutment Foundations (see note below) . Tons	2·8	3·2	3·6	4 0	4 4	6 0
Spans in Feet.	**15**	**20**	**25**	**30**	**40**	**50**
12. C for 1 : 2 : 4 P. C. Concrete. Equation (3).	16"	18"	20"	21"	24'	27"
13. C for C.D.V. Stone in 1 : 2 P.C. Mortar.	16"	18"	20"	21"	24"	27"
14. 1½ (3) for H.D.V. Stone in 1 : 3 P.C Mortar.	18"	20"	22"	24"	..	..
15. 1½ (3) for H.D.V. Stone in Lime Mortar . . . . . C=	18"	22"	24"	26"	30"	36"
16. 1½(3) for Hard Burnt Bricks in Lime . . . . . C—	18"	22½"	27"	27"	31½"	36"
17. E for r/S = ½, ⅓, ¼ (h less than 1½ b)	4' 3"	5' 0"	5' 9"	6' 6"	8' 0"	9' 6"
18. E for r/S = ⅙ (h less than 1½ b) .	4' 6"	5' 4"	6' 2"	7' 0"	8' 7"	10' 3"
19 P for P.C. Concrete or H.D. Stone in Lime.	2' 9"	3' 0"	3' 6"	4' 0"	5' 0"	7' 0"
20. P for Brick in Lime Mortar . .	3' 4½"	3' 9"	4' 6"	4' 10½"	6' 9"	8' 6"
21. G for 1 : 3 : 6 P.C. Concrete . .	1' 9"	2' 0"	2' 0"	2' 6"	2' 6"	2' 6"
22. G for best Lime Concrete . .	2' 6"	2' 9"	3' 0"	3' 3"	3' 9"	4' 0"
23. Load for Pier Foundations (see note below) . . Tons	11	16	21	26	42	62
24. Load for Abutment Founda-						

PLATE XVII.

ABUTMENTS FOR SLAB, R.S.J., R.C. BEAM AND TRUSS SPANS.



ABUTMENTS FOR R.C. SLABS, BEAMS R.S.J. AND TROUGHING SPANS. | ABUTMENTS FOR GIRDERS AND TRUSSES.

Clear Span Ft	10'	20'	30'	40'	50'	75'	100'	150'	200'
P.C. Masonry a=	15"	18"	22"	26"	30"	32"	34"	37"	40"
Lime Masonry a=	18"	24"	27"	32"	37"	40"	42"	46"	50"

All Dimensions are the minimum allowable. Batter may be stepped

ARRANGEMENT OF ABUTMENTS FOR ARCH, BEAM AND TRUSS BRIDGES.

COMBINED PLAN OF RETURNED AND WING WALLS WITH VERTICAL AND BATTERED FACES.

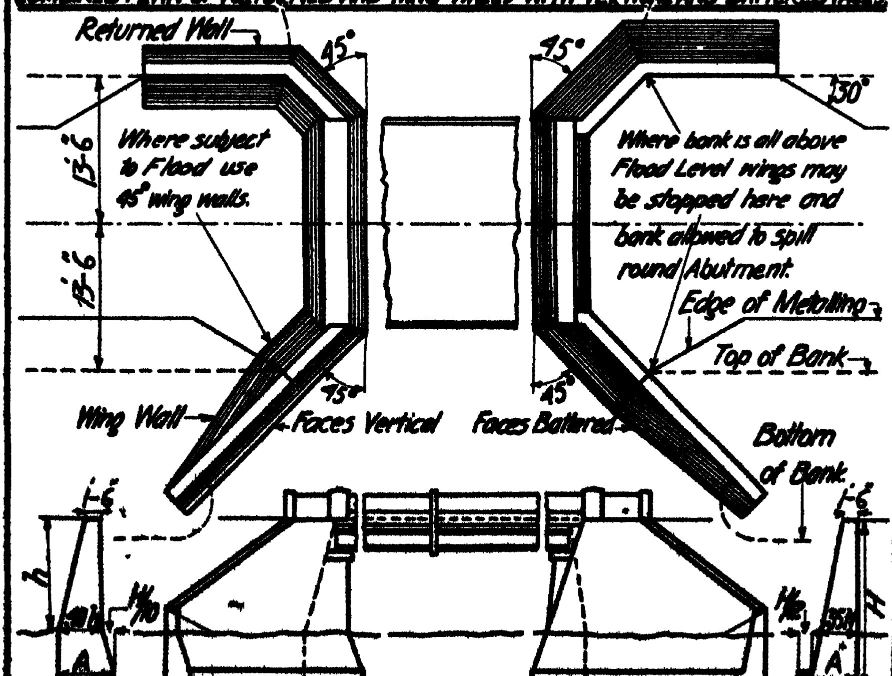


PLATE XVIII.

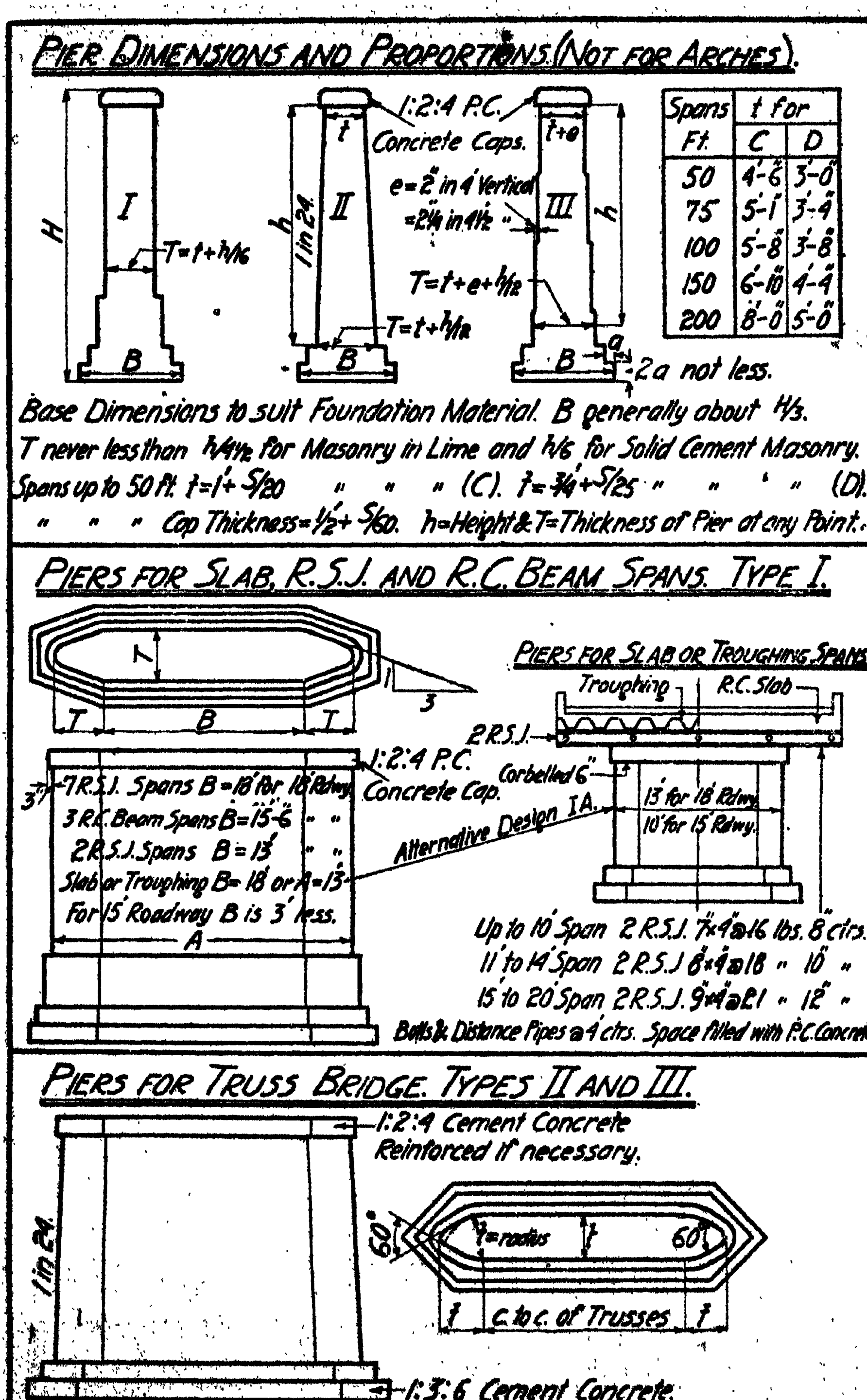
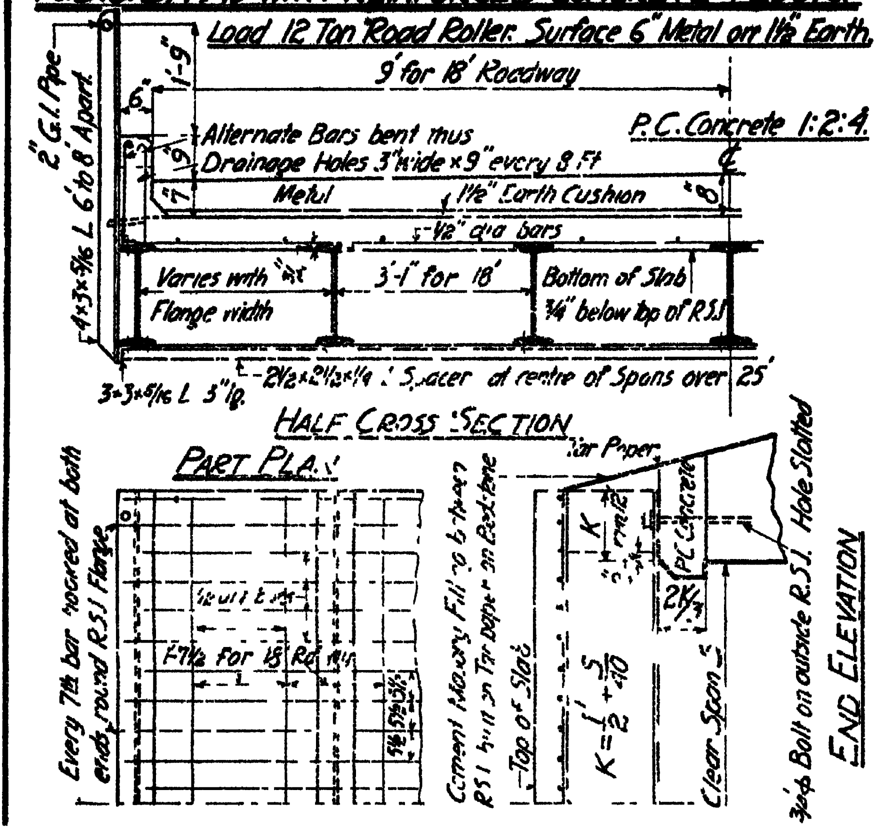

PIER DIMENSIONS AND PROPORTIONS (NOT FOR ARCHES).
H
I
T=t+h/16
B
1:2:4 P.C.
Concrete Caps.
t
h
1 in 24.
II
T=t+h/12
B
t+e
e=2" in 4' Vertical
=2¼" in 4½' "
III
T=t+e+h/12
h
B
a
2a not less.
Spans Ft. | t for C | t for D
50 | 4'-6" | 3'-0"
75 | 5'-1" | 3'-4"
100 | 5'-8" | 3'-8"
150 | 6'-10" | 4'-4"
200 | 8'-0" | 5'-0"
Base Dimensions to suit Foundation Material. B generally about H/3.
T never less than h/4½ for Masonry in Lime and h/6 for Solid Cement Masonry.
Spans up to 50 ft. t=1'+S/20 " " " (C). t=¾'+S/25 " " " " (D).
" " " Cap Thickness=½'+S/60. h=Height & T=Thickness of Pier at any Point.
PIERS FOR SLAB, R.S.J. AND R.C. BEAM SPANS. TYPE I.
h
1
3
T
B
T
1:2:4 P.C.
Concrete Cap.
3"
7 R.S.J. Spans B=18' for 18' Rdwy.
3 R.C. Beam Spans B=15'-6" " "
2 R.S.J. Spans B=13' " "
Slab or Troughing B=18' or A=13'
For 15' Roadway B is 3' less.
A
Alternative Design IA.
PIERS FOR SLAB OR TROUGHING SPANS
Troughing
R.C. Slab
2 R.S.J.
Corbelled 6"
13' for 18' Rdwy.
10' for 15' Rdwy.
Up to 10' Span 2 R.S.J. 7"×4" @ 16 lbs. 8" ctrs.
11' to 14' Span 2 R.S.J. 8"×4" @ 18 " 10" "
15' to 20' Span 2 R.S.J. 9"×4" @ 21 " 12" "
Bolts & Distance Pipes @ 4' ctrs. Space filled with P.C. Concrete
PIERS FOR TRUSS BRIDGE. TYPES II AND III.
1:2:4 Cement Concrete
Reinforced if necessary.
1 in 24.
60°
t=radius
t
60°
t
C. to C. of Trusses
t
1:3:6 Cement Concrete.

PLATE XIX.

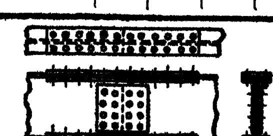
R.S.J. SPANS WITH REINFORCED CONCRETE FLOOR.



18' AND 15' ROADWAY 6½ SLAB ½ BARS 5 CENTRES 10 ROADWAY 6" SLAB ½ BARS ½ CENTRES
SPANS OVER 30 FT HAVE ½" BEARING PLATES

CLEAR SPAN FT		8	10	12	14	16	18	20	22	24	26	28	30
18' and 15' Roadway.	Modulus one R S J	13 7	15 0	22 1	27 8	32 8	30 2	4[illegible] 4	[illegible] 1	64 2	72 [illegible]	81 4	90 6
	Section R S J	8 @ 13 lbs	9" @ 21 lbs.	10 @ 25 lbs.	12 @ 30 lbs.	12 @ 30 lbs.	13" @ 35 lbs.	14' @ 40 lbs.	15 @ 42 lbs.	15 @ 45 lbs	16' @ 50 lbs.	16" @ 55 lbs.	18" @ 55 lbs.
CLEAR SPAN FT		32	34	36	38	40	42	44	46	48	50	52	54
18' and 15' Roadway.	Modulus one R.S.J	100	110	122	1[illegible]3	144	155	167	178	191	203	217	229
	Section R S J	20' @ 65 lbs	20" @ 65 lbs.	20" @ 65 lbs.	22" @ 75 lbs.	22" @ 75 lbs.	22" @ 75 lbs.	24" @ 90 lbs.	24" @ 90 lbs.	24" @ 90 lbs.	24" @ 90 lbs.	24" @ 100 lbs.	24" @ 100 lbs.

New British Standard Sections are given if not available, use Sections with required Modulus
For 15' Roadway use 6 lines R S J @ 2'-11" Centres
For 10' Roadway use 4 lines R S J @ 3'-6" Centres
and multiply Modulus by $\frac{\text{Spacing of R S J in ft}}{3\ 05'}$

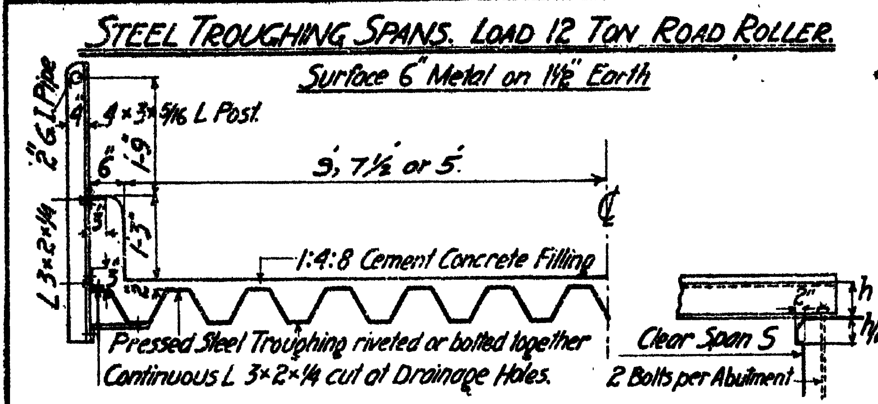


R.S.J. SPLICE

TABLE XX.

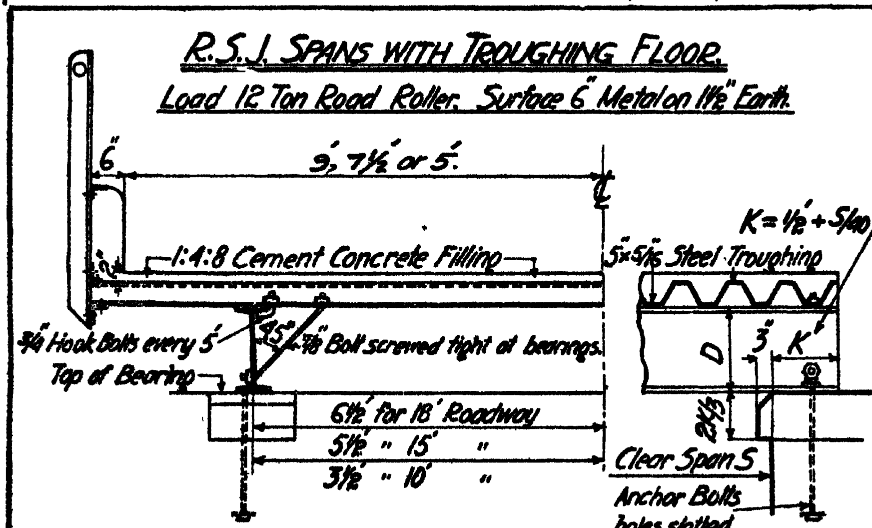
Pressed Steel Troughing.							
Section depth × thickness.	Weight per sq. foot.	Section Modulus per ft. wide.	Moment per foot wide $f=6\frac{1}{2}$ Tons.	Section depth × thickness.	Weight per sq. Foot.	Section Modulus per ft. wide.	Moment per foot wide $f=6\frac{1}{2}$ Tons.
Ins.	lbs.	Ins.^{3}	Ft. lbs.	Ins.	lbs.	Ins.^{3}	Ft. lbs.
$4 \times \frac{5}{16}$	17·4	6·0	7,300	$9 \times \frac{3}{8}$	28·1	19·9	24,100
$\frac{3}{8}$	20·8	7·2	8,720	$\frac{7}{16}$	32·6	23·2	28,100
$5 \times \frac{5}{16}$	17·7	6·8	8,300	$\frac{1}{2}$	37·1	26·5	32,100
$\frac{3}{8}$	21·3	8·2	10,000	$10 \times \frac{3}{8}$	25·7	20·2	24,400
$6 \times \frac{5}{16}$	19·2	8·6	10,400	$\frac{7}{16}$	29·8	23·6	28,500
$\frac{3}{8}$	23·0	10·6	12,800	$\frac{1}{2}$	33·9	27·0	32,700
$6\frac{1}{2} \times \frac{5}{16}$	19·2	10·0	12,100	$12 \times \frac{3}{8}$	25·9	22·1	26,800
$\frac{3}{8}$	23·0	12·0	14,500	$\frac{7}{16}$	30·0	25·8	31,200
$7\frac{1}{2} \times \frac{3}{8}$	25·8	15·5	18,800	$\frac{1}{2}$	34·2	29·6	35,800
$\frac{7}{16}$	30·0	18·1	21,900	$15 \times \frac{3}{8}$	28·4	31·0	37,500
$\frac{1}{2}$	34·1	20·6	25,000	$\frac{7}{16}$	33·0	36·1	43,600
				$\frac{1}{2}$	37·5	41·2	49,900

PLATE XXI.



HALF CROSS SECTION. END ELEVATION.

Clear Span Feet.	2' to 10'	12'	14'	16'	18'	20'	22'
Modulus per ft. wide—required .	..	7·.	8·6	10·2	12·2	14·5	17·0
Depth × Thickness . . .	4" × $\frac{3}{16}$"	5" × $\frac{5}{16}$"	6" × $\frac{5}{16}$"	6½" × $\frac{5}{16}$"	7½" × $\frac{3}{8}$"	7½" × $\frac{3}{8}$"	9" × $\frac{3}{8}$"

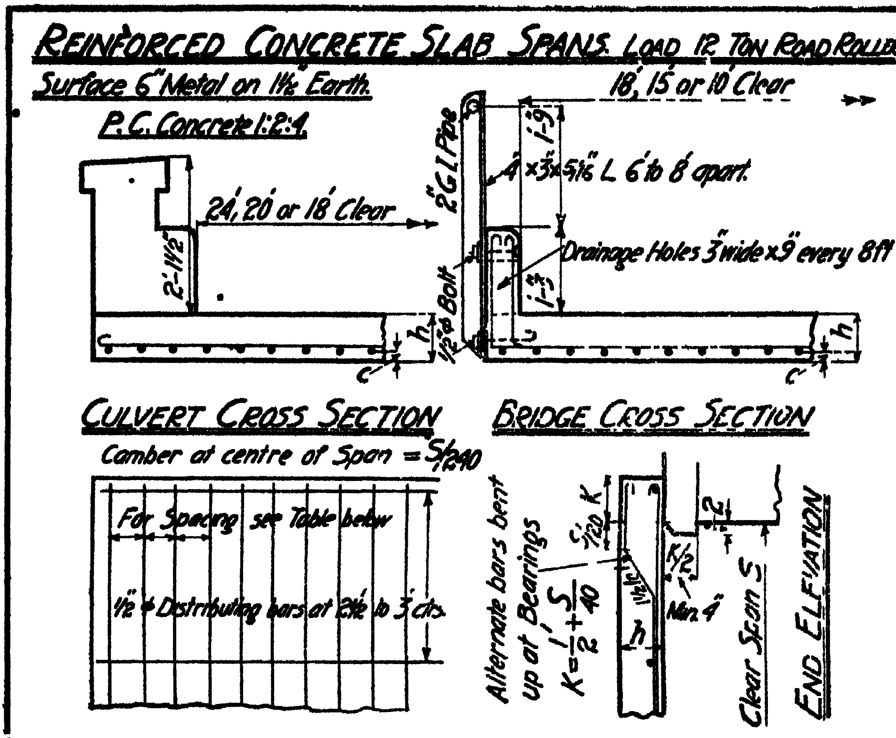


HALF CROSS SECTION. END ELEVATION.

Clear Span—— Feet.	12'	14'	16'	18'	20'	22'	24'	26'
Modulus required one R.S.J.	72'	89'	105'	126	152	176	205	232
Section R.S.J.	16" @ 50 lbs.	18" @ 55 lbs.	20" @ 65 lbs.	20" @ 65 lbs.	22" @ 75 lbs.	24" @ 90 lbs.	24" @ 90 lbs.	24" @ 100 lbs.

New British Standard Sections are given, if not available use a R.S.J. with Modulus required.
For 18' Roadway. Troughing 5" × $\frac{5}{16}$" R.S.J. as above.
For 15' Roadway. Troughing 4" × $\frac{3}{16}$". Use R.S.J. with Modulus given above × ·94.
For 10' Roadway. Troughing 4" × $\frac{3}{16}$". Use R.S.J. with Modulus given above × ·84.

PLATE XXII.



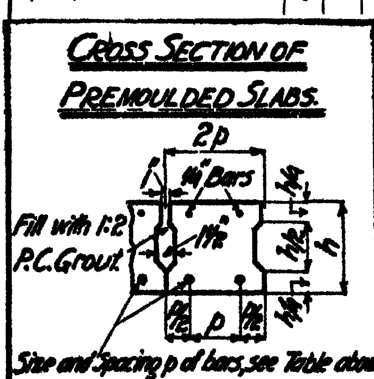
PART PLAN

[illegible]

CULVERT SLABS [illegible] H [illegible] Slab and S Clear Span in Feet.

For Slabs given in Table, Maximum $H = \frac{80}{S}$ For greater [illegible] $h = \frac{S}{2\cdot 8}\sqrt{H}$

Clear Span Feet	2	3	4	5	6	7	8	9	10	11	12	13	14	15
[illegible] h [illegible]	[illegible]	[illegible]		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible] centre of [illegible]	1	1	[illegible]	1	[illegible]	1	[illegible]	[illegible]	[illegible]	1	[illegible]	1	[illegible]	[illegible]
[illegible]		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Spacing of Bar	[illegible]		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]



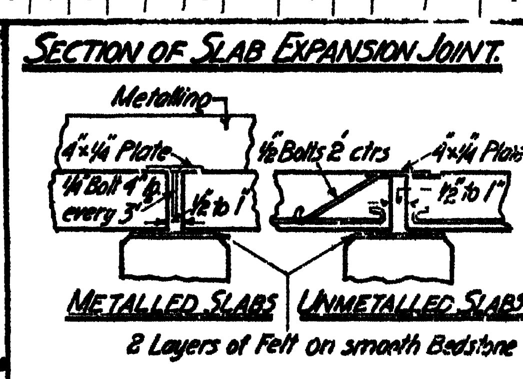


PLATE XXIII.

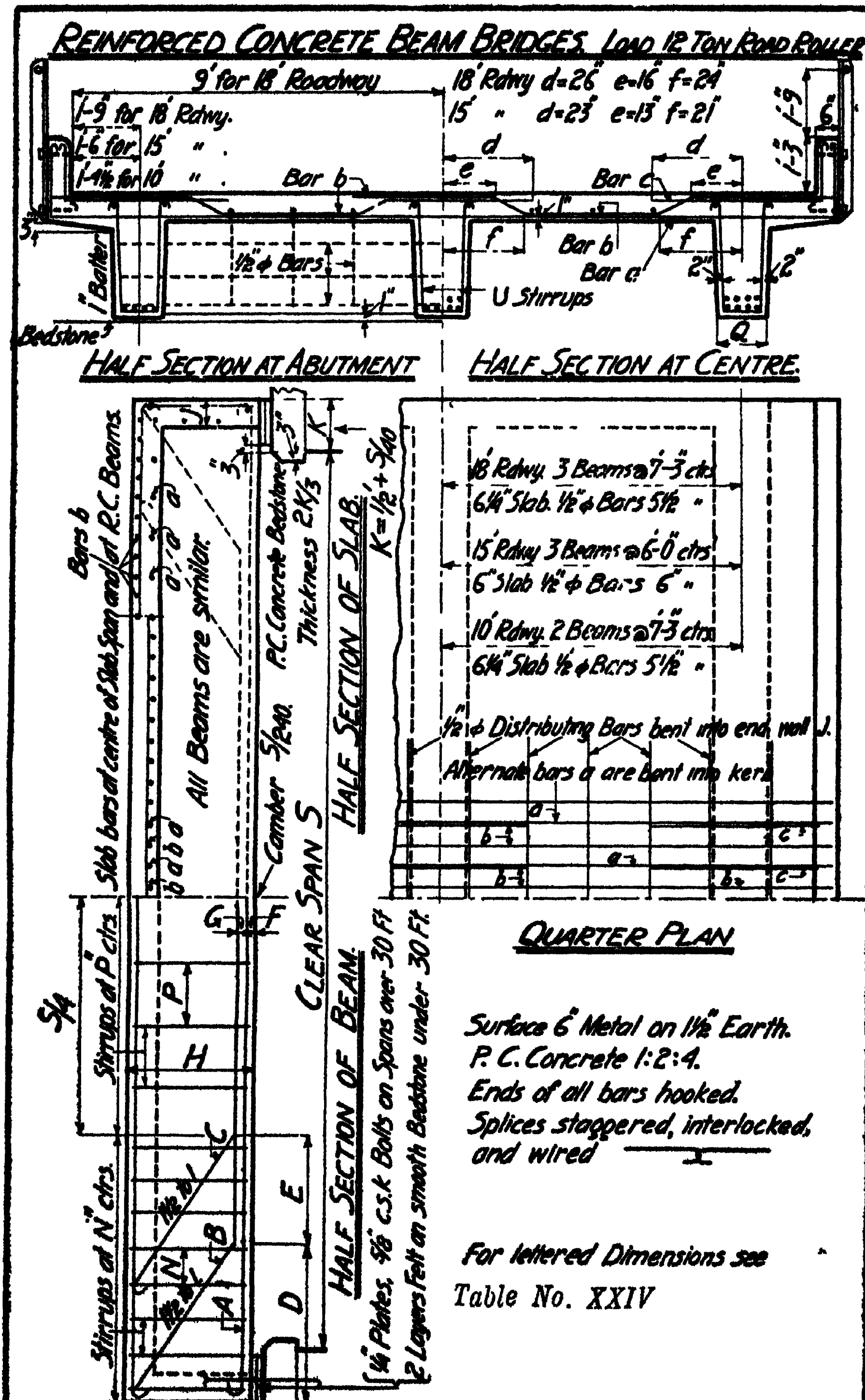
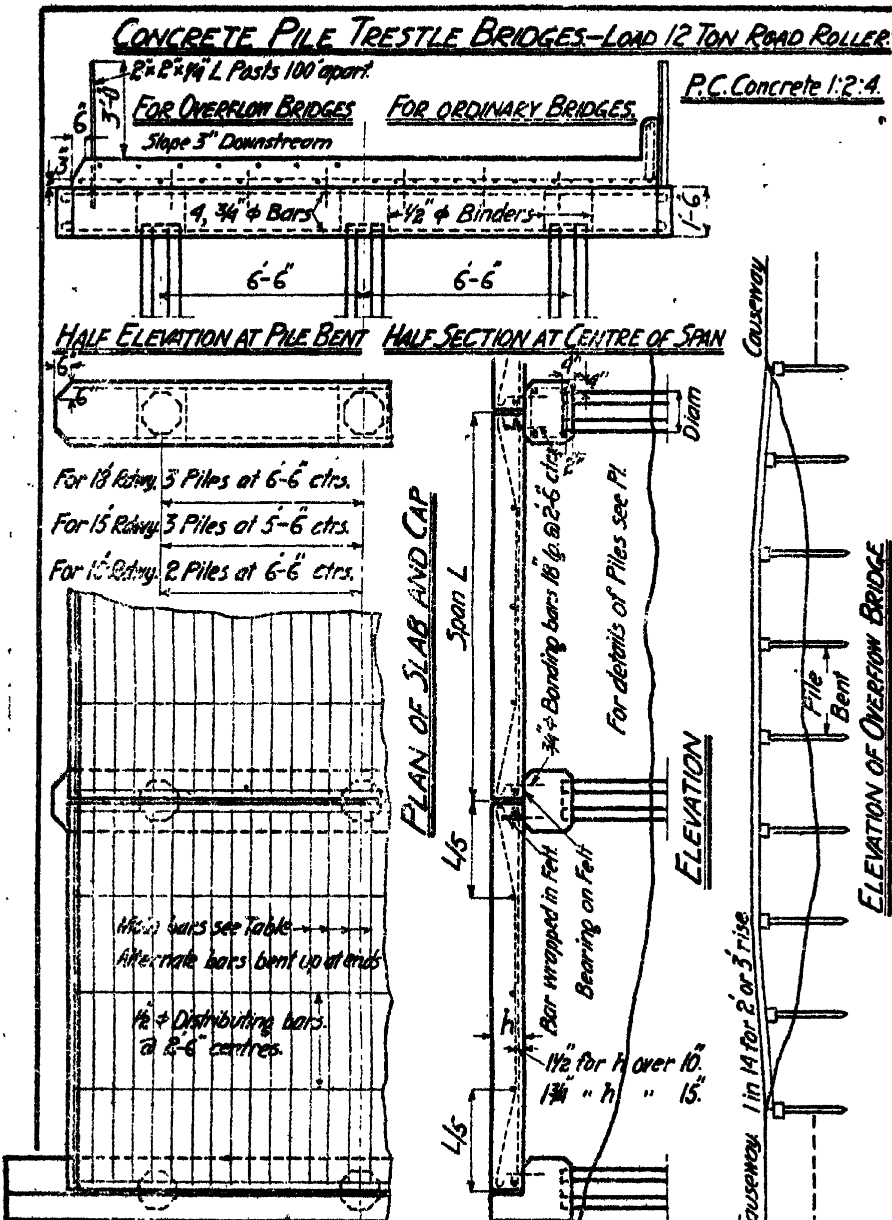

REINFORCED CONCRETE BEAM BRIDGES. LOAD 12 TON ROAD ROLLER
9' for 18' Roadway
18' Rdwy d=26" e=16" f=24"
15' " d=23" e=13" f=21"
1-9" for 18' Rdwy.
1-6" for 15' " .
1-4½" for 10' " .
Bar b
Bar c
Bar b
Bar a
½" ϕ Bars
U Stirrups
Bedstone
HALF SECTION AT ABUTMENT
HALF SECTION AT CENTRE.
18' Rdwy. 3 Beams @ 7'-3" ctrs.
6¼" Slab. ½" ϕ Bars 5½" "
15' Rdwy 3 Beams @ 6'-0" ctrs.
6" Slab ½" ϕ Bars 6" "
10' Rdwy. 2 Beams @ 7'-3" ctrs.
6¼" Slab ½" ϕ Bars 5½" "
½" ϕ Distributing Bars bent into end wall J.
Alternate bars a are bent into kerb
Bars b
Slab bars at centre of Slab Span and at R.C. Beams
All Beams are similar.
Camber S/240
P.C. Concrete Bedstone Thickness 2K/3
CLEAR SPAN S
HALF SECTION OF SLAB.
K = ½' + S/40
Stirrups at P" ctrs.
Stirrups at N" ctrs.
S/4
HALF SECTION OF BEAM.
¼" Plates, 5/8" c.s.k Bolts on Spans over 30 Ft
2 Layers Felt on smooth Bedstone under 30 Ft.
QUARTER PLAN
Surface 6" Metal on 1½" Earth.
P. C. Concrete 1:2:4.
Ends of all bars hooked.
Splices staggered, interlocked, and wired
For lettered Dimensions see
Table No. XXIV

TABLE XXIV.

Dimensions and reinforcement for reinforced concrete reams.

Clear Span Ft.	10	12	14	16	18	20	22	24	20	28	30	32	34	36	38	40	42	44	46	48	50
18', 15' or 10' Roadway.																					
Depth . . H	1'-7"	1'-9"	1'-11"	2'-1"	2'-3"	2'-8"	2'-10"	3'-0"	3'-2"	3'-4"	3'-6"	3'-8"	3'-10"	4'-0"	4'-2"	4'-4"	4'-6"	4'-8"	4'-10"	5'-0"	5'-2"
Width . . Q	1'-2"	—	—	—	—	—	—	—	—	—	—	1'-2"	1'-4"	—	—	—	—	1'-4"	1'-6"	1'-6"	1'-6"
Bearing . . K	1'-0"	—	—	—	—	1'-0"	1'-1"	1'-1"	1'-2"	1'-2"	1'-3"	1'-4"	1'-4"	1'-5"	1'-5"	1'-6"	1'-7"	1'-7"	1'-8"	1'-8"	1'-9"
Ballast Wall . J	0	—	Fill J with Masonry	—	—	—	—	0	6"	—	—	—	—	6"	9"	—	—	—	—	—	9"
Position of Bars F	2"	—	—	—	2"	2½"	—	—	—	—	2½"	3"	—	—	—	—	—	—	—	—	3"
Position of Bars G	0	—	—	—	0	2½"	—	—	—	—	2½"	3"	—	—	—	—	—	—	—	—	3"
Bends in Bars D	2'-2"	2'-5"	2'-9"	3'-1"	3'-4"	3'-8"	4'-0"	4'-4"	4'-8"	5'-0"	5'-4"	5'-8"	6'-0"	6'-4"	6'-8"	7'-0"	7'-4"	7'-8"	8'-0"	8'-4"	8'-8"
Bends in Bars E	0	—	—	—	0	2'-2"	2'-4"	2'-6"	2'-9"	3'-0"	3'-2"	3'-4"	3'-6"	3'-9"	3'-11"	4'-2"	4'-4"	4'-7"	4'-9"	5'-0"	5'-2"
Diameter of Stirrups	½"	—	—	—	—	—	½"	⅝"	—	—	—	—	—	—	—	—	—	—	—	—	⅝"
Spacing of Stirrups N	6"	—	—	—	—	—	6"	10"	—	—	—	—	—	—	—	10"	12"	—	—	—	12"
Spacing of Stirrups P	12"	—	—	—	—	—	12"	20"	—	—	—	—	—	—	—	20"	24"	—	—	—	24"
18' or 10' Roadway.																					
One Beam: Area of Steel sq. ins.	2·8	3·2	3·6	4·0	4·1	4·9	5·4	5·9	6·4	6·9	7·4	7·9	8·5	9·0	9·5	10·2	10·7	11·2	11·9	12·5	13·1
2 Bars C dia.	0	One layer only	—	—	0	⅞"	⅞"	⅞"	1"	1"	1"	1⅛"	1⅛"	1⅛"	1¼"	1¼"	1¼"	1¼"	1⅜"	1⅜"	1⅜"
2 Bars B dia.	⅞"	1"	1"	1⅛"	1⅛"	⅞"	⅞"	1"	1"	1"	1⅛"	1⅛"	1⅛"	1⅛"	1¼"	1¼"	1¼"	1⅜"	1⅜"	1⅜"	1⅜"
2 Bars A dia.	1"	1"	1⅛"	1⅛"	1¼"	⅞"	1"	1"	1"	1⅛"	1⅛"	1⅛"	1⅛"	1¼"	1¼"	1¼"	1⅜"	1⅜"	1⅜"	1⅜"	1½"
2 Bars A dia.	0	—	—	—	0	⅞"	1"	1"	1"	1⅛"	1⅛"	1⅛"	1¼"	1¼"	1¼"	1⅜"	1⅜"	1⅜"	1⅜"	1½"	1½"
15' Roadway.																					
One Beam: Area of steel sq. ins.	2·6	2·9	3·3	3·6	4·0	4·4	4·9	5·4	5·8	6·3	6·8	7·2	7·7	8·2	8·7	9·2	9·8	10·2	10·8	11·4	11·9
2 Bars C dia.	0	One layer only	—	—	0	⅞"	⅞"	⅞"	⅞"	1"	1"	1"	1⅛"	1⅛"	1⅛"	1⅛"	1¼"	1¼"	1¼"	1¼"	1⅜"
2 Bars B dia.	⅞"	1"	1"	1"	1⅛"	⅞"	⅞"	⅞"	1"	1"	1"	1⅛"	1⅛"	1⅛"	1¼"	1¼"	1¼"	1¼"	1¼"	1⅜"	1⅜"
2 Bars A dia.	1"	1"	1⅛"	1⅛"	1⅛"	⅞"	⅞"	1"	1"	1"	1"	1⅛"	1⅛"	1⅛"	1¼"	1¼"	1¼"	1¼"	1⅜"	1⅜"	1⅜"
2 Bars A dia.	0	—	—	—	0	⅞"	⅞"	1"	1"	1"	1⅛"	1⅛"	1⅛"	1¼"	1¼"	1¼"	1¼"	1⅜"	1⅜"	1⅜"	1⅜"

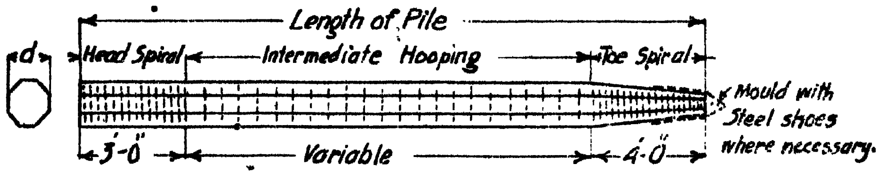
PLATE XXV.



Span L	Slab Thickness h	Slab Spacing of Main Bars	Piles Load on One. lbs.	Piles Diameter	Span L	Slab Thickness h	Slab Spacing of Main Bars	Piles Load on One. lbs.	Piles Diameter
12'	10½"	¾" φ @ 7¼"	18,700	12"	18'	14¾"	⅞" φ @ 7"	26,100	14"
14'	12"	" " 6¼"	21,200	12"	20'	16½"	" " 6"	28,700	14"
16'	13¼"	⅞" φ @ 7½"	23,700	12"					

TABLE XXVI.

REINFORCED CONCRETE PILES.



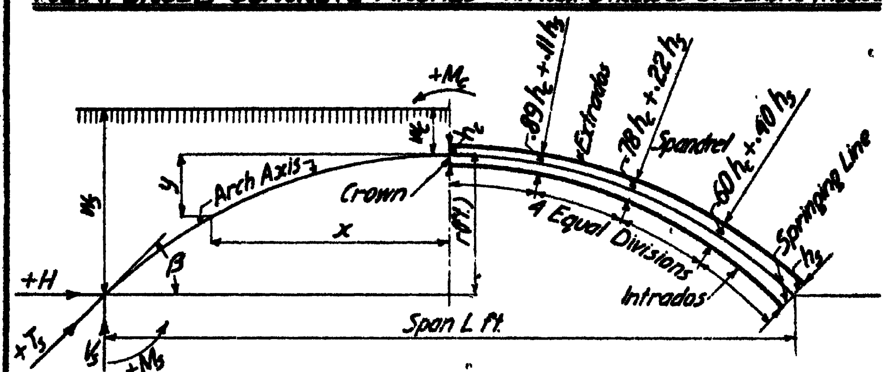
CONCRETE MIXTURE 1 : 2 : 4.

Maximum Length of Pile.	Outside Diameter.	Core Diameter.	Round Bar Verticals.		Head Spiral.		Hooping.		Toe Spiral.		Max. Load See note below.	Load for average conditions.
					Rounds.	Pitch.	Rounds.	Spacing.	Rounds.	Pitch.		
Feet.	Inches.	Inches.	No.	Size.	Size.	Inches.	Size.	Inches.	Size.	Inches.	lbs.	lbs.
20	10	7	6	½	¼	2	¼	8	¼	1½	22,000	15,000
25	12	9	6	⅝	¼	2	¼	10	¼	1½	36,000	25,000
30	14	10	6	¾	¼	2	¼	12	¼	1½	45,000	30,000
35	16	12	6	⅞	1⅜	2	¼	12	¼	1½	64,000	45,000
40	18	14	8	⅞	⅜	2	⅜	12	⅜	1½	87,000	60,000

NOTE.—Reduce the Maximum Load according to the unsupported length of Pile and also in accordance with Formula in Para.

PLATE XXVII.

REINFORCED CONCRETE ARCHES:—APPROX. STRESSES BY ELASTIC THEORY



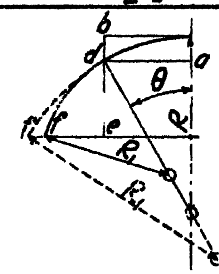
$$\tan\beta = \frac{4r}{L^2(U+5)}\left[6L+2L(U-1)\right] \qquad y=\frac{4r}{L^2(U+5)}\left[6x^2+(U-1)\frac{4x^4}{L^2}\right] \qquad U=\frac{w_s}{w_c}$$

RADII FOR 3 CENTERED CURVE

$$R=\frac{ad^2+bd^2}{2bd} \qquad \sin\theta=\frac{ad}{R}$$

$$R_1=\frac{ef^2+ed^2}{2(ed\cos\theta - ef\sin\theta)}$$

Note. R_1 may be greater or less than R.



Max. Stresses due to	H	M_c	T_s	M_s	V_s
1. Dead Load . .	$\frac{w_c L^2}{48 r}(U+5)$	$\pm\frac{H h_c}{40}$	H sec. β	$\pm\frac{T_s h_s}{40}$	$\frac{w_c L}{6}(U+2)$
2. Live Load for Max. $\pm M_c$	$\frac{w L^2}{16 r}$	$\pm\frac{w L^2}{110}$			
3. Live Load for Max. $+M_s$	$\frac{w L^2}{10 r}$	Open Spandrel——→		$+\frac{w L^2}{40}$	$\frac{w L}{5}$
		..	H sec. β	..	
		Filled Spandrel——→		$+\frac{w L^2}{30}$	
4. Live Load for Max. $-M_s$	$\frac{w L^2}{30 r}$	..	H sec. β	$-\frac{w L^2}{50}$	$\frac{w L}{3}$
5. Temperature $\pm t°$	$\pm\frac{4500\, b\, h^3\, t°}{r^2}$	$\mp\frac{H r}{4}$	$\pm\frac{H}{\sec.\beta}$	$\pm\frac{3 H r}{4}$	..
6. Arch Shortening .	$-\left(1+\frac{\cdot3 h_s^2}{h_c^2}\right)\frac{H_a h_c^2}{r^2 \sec.\beta}$	$+\frac{H r}{4}$	$-\frac{H}{\sec.\beta}$	$-\frac{3 H r}{4}$	..
7. Live Load over whole span.	$\frac{w L^2}{8 r}$	..	..	..	$\frac{w L}{2}$

For Crown take items 1, 2, 5, 6. ALT. [illegible]

PLATE XXIX.

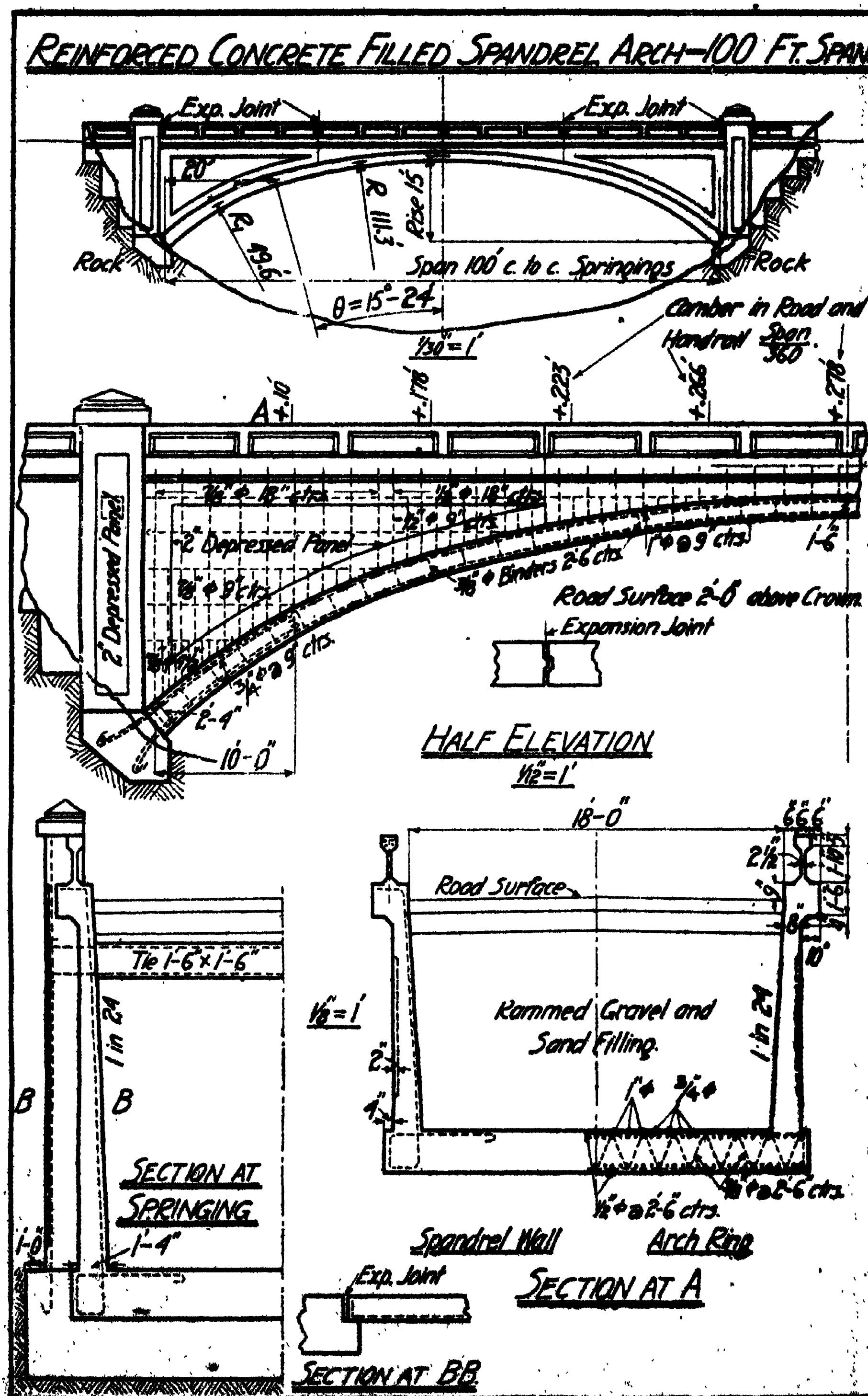
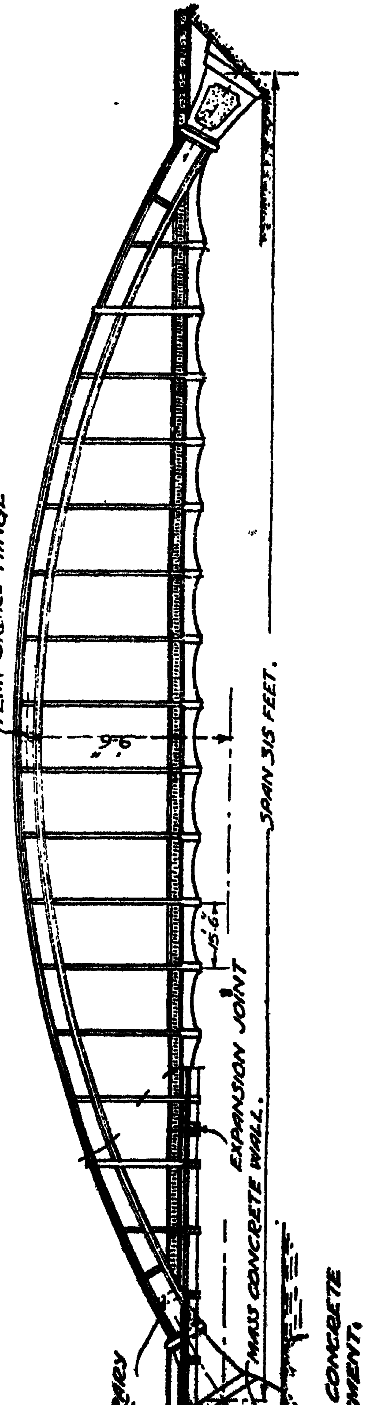

REINFORCED CONCRETE FILLED SPANDREL ARCH—100 FT. SPAN
Exp. Joint
Exp. Joint
Rise 15'
R 111.3'
R1 49.6'
Rock
Rock
Span 100' c. to c. Springings
θ=15°-24
1/30"=1'
Camber in Road and Handrail Span/360
A
2" Depressed Panel
2" Depressed Panel
Binders 2'-6 ctrs.
Road Surface 2'-6 above Crown
Expansion Joint
10'-0"
HALF ELEVATION
1/12"=1'
18'-0"
Road Surface
Tie 1'-6"×1'-6"
1 in 24
1 in 24
1/8"=1'
Rammed Gravel and Sand Filling.
B
B
SECTION AT SPRINGING
1'-4"
1'-0
Spandrel Wall
Arch Ring
SECTION AT A
Exp. Joint
SECTION AT BB

PLATE XXX.

Reinforced Concrete Bowstring Bridge.

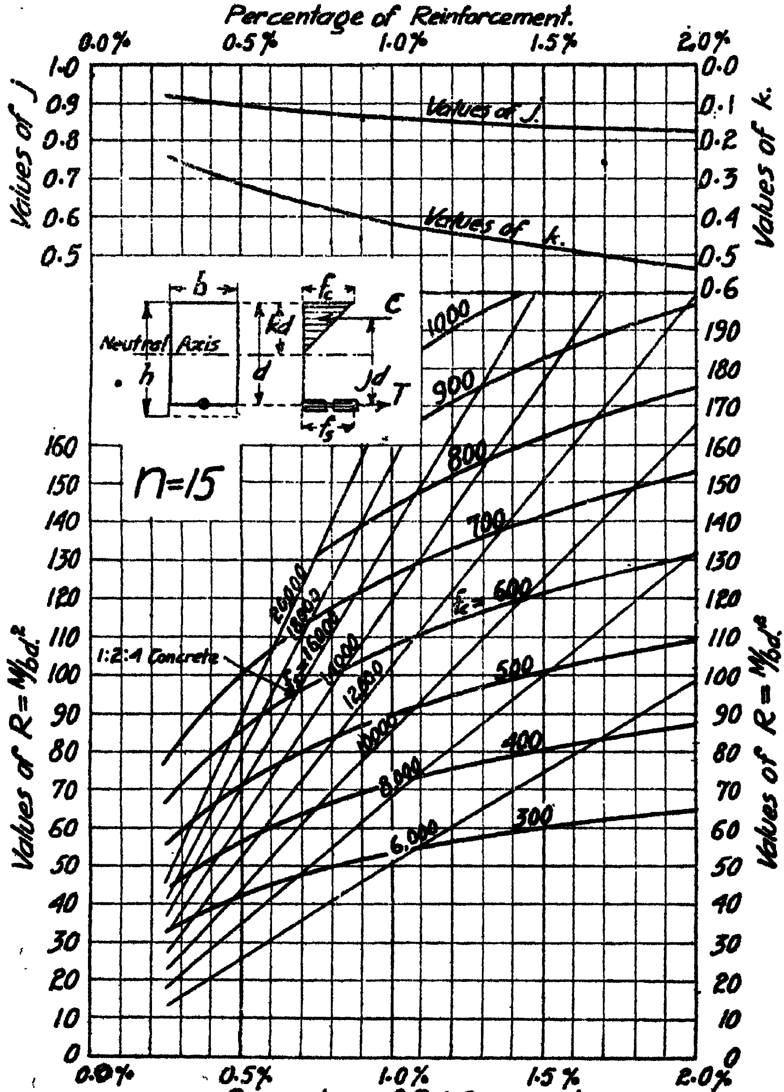


Part Longitudinal Section.

PLATE XXXI.

RECTANGULAR BEAMS AND SLABS.

T BEAMS WHERE kd IS EQUAL TO OR LESS THAN t.



$p = \frac{A_s}{bd}$ (1) $k = \sqrt{2pn + p^2n^2} - pn$ (2) $= \frac{1}{1 + \frac{f_s}{nf_c}}$ (3) $j = 1 - \frac{k}{3}$ (4)

$M_s = f_s A_s j d$ (5) $= f_s p j b d^2$ (6) $M_c = \frac{1}{2} f_c k j b d^2$ (7) $\frac{M}{bd^2} = R$ (8)

$A_s = \frac{M}{f_s j d}$ (9)

Balanced Reinforcement $M_s = M_c$. $p = \frac{1}{2f_s / f_c \ldots}$ (10) $d = \sqrt{\frac{M}{Rb}}$ (11)

TABLE XXXII.

Double Reinforced Beams.

Compressive and Tensile Steel Percentages for Balanced Design with Values of $\frac{M}{bd^2}$

$f_c = 600$, $f_s = 16{,}000$ lbs. per Sq. Inch.
d'=Depth to Compressive Steel.
d'=Depth to Tensile Steel.

°/₀ Compressive Steel.	$d'/d = \cdot 05$		$d'/d = \cdot 10$		$d'/d = \cdot 15$	
	°/₀ Tensile Steel.	$R = \frac{M}{bd^2}$	°/₀ Tensile Steel.	$R = \frac{M}{bd^2}$	°/₀ Tensile Steel.	$R = \frac{M}{bd^2}$
·0	·675	95	·675	95	·675	95
·2	·77	110	·75	107	·73	104
·4	·87	125	·84	119	·80	113
·6	·97	140	·92	130	·86	122
·8	1·06	153	1·00	142	·93	131
1·0	1·15	168	1·08	154	1·00	140
1·2	1·25	183	1·16	165	1·06	148
1·4	1·35	198	1·25	177	1·13	157
1·6	1·45	213	1·33	189	1·20	166
1·8	1·65	227	1·41	201	1·26	175
2·0	1·64	242	1·50	213	1·33	184

PLATE XXXIII.

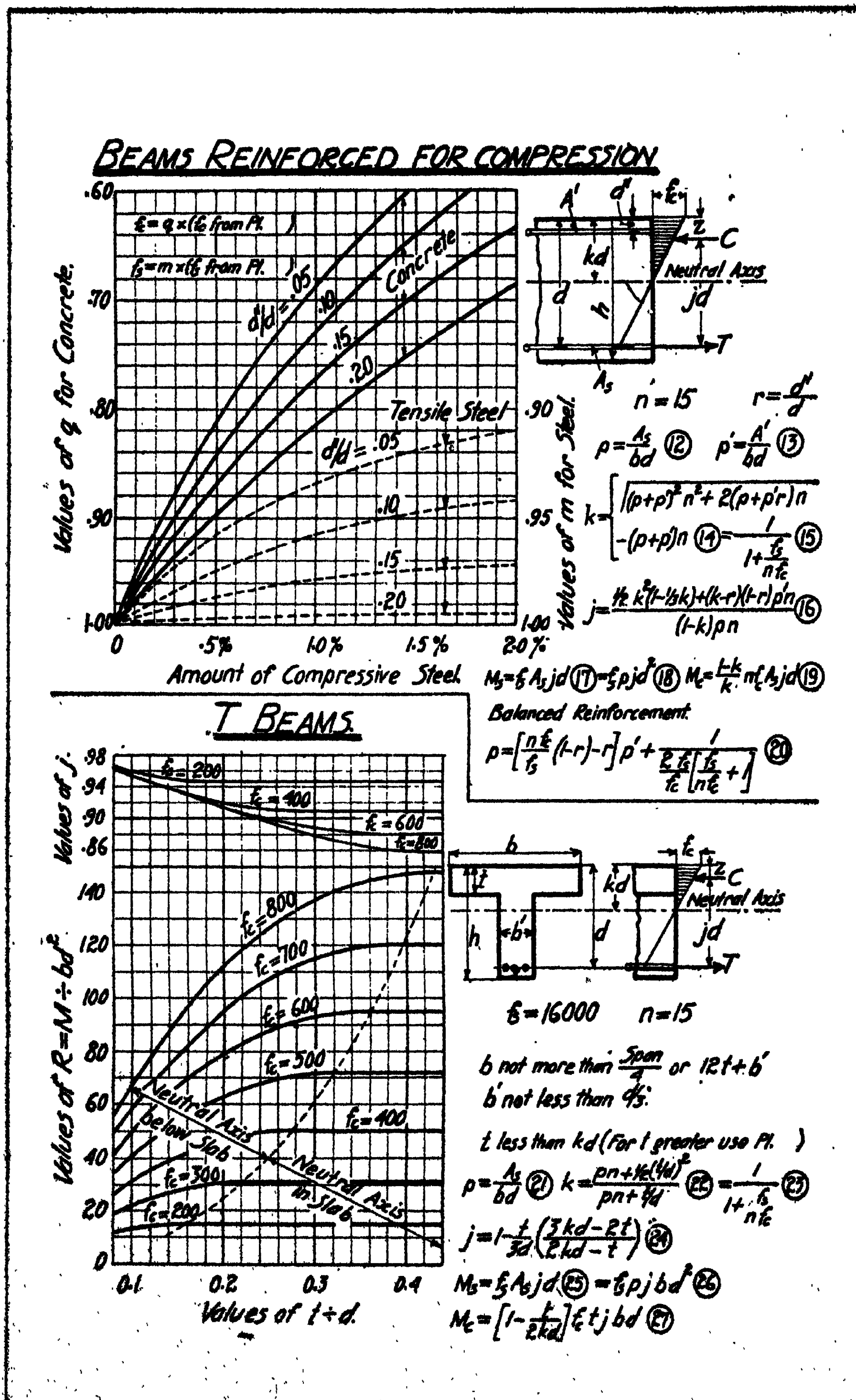


PLATE XXXIV.

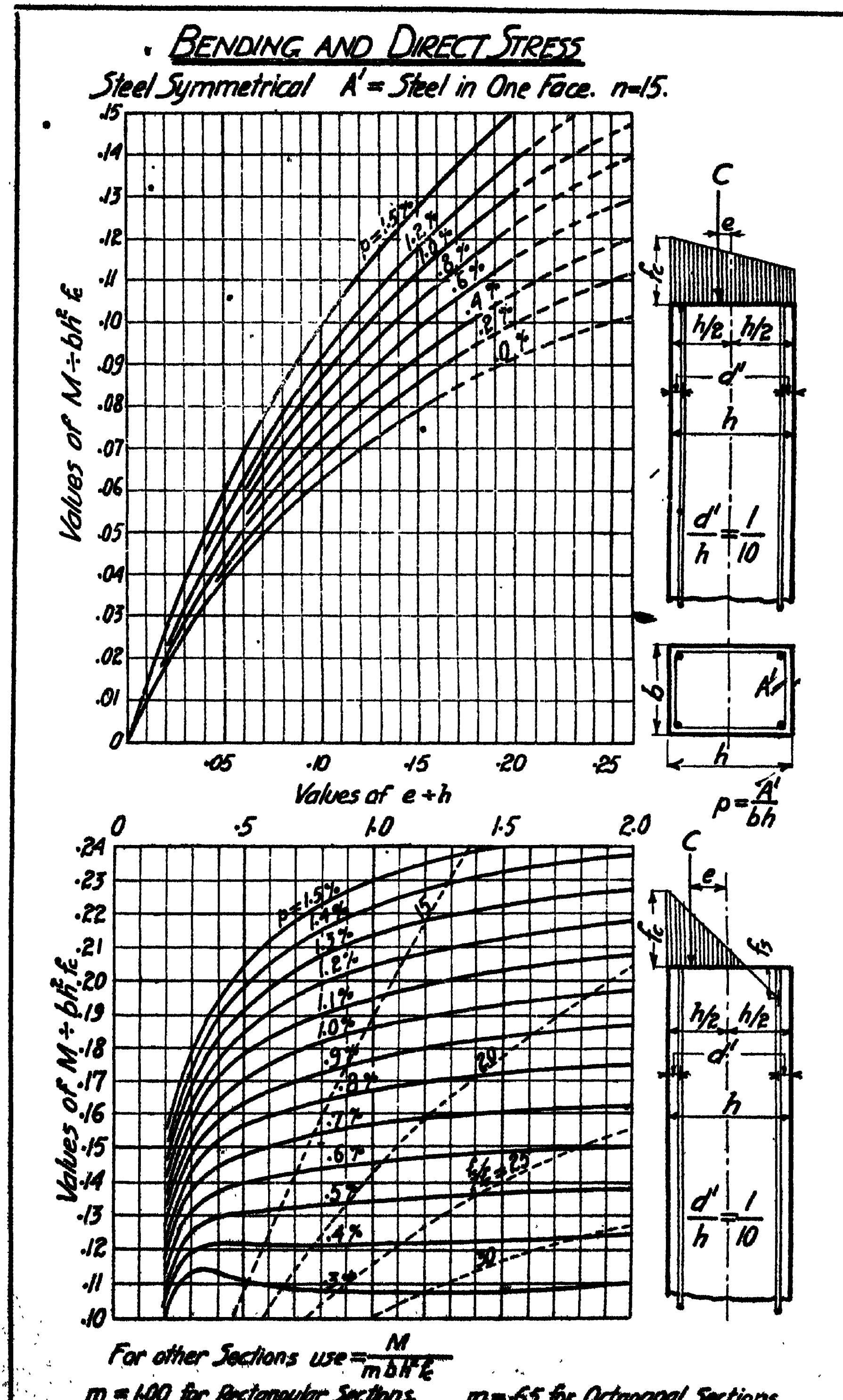

Bending and Direct Stress
Steel Symmetrical A' = Steel in One Face. n=15.
Values of M ÷ bh² fc
Values of e ÷ h
p=1.5%
1.2%
1.0%
.8%
.6%
.4%
.2%
0%
C
e
fc
h/2 h/2
d'
h
d'/h = 1/10
b
A'
p = A'/bh
Values of M ÷ bh² fc
p=1.5%
1.4%
1.3%
1.2%
1.1%
1.0%
.9%
.8%
.7%
.6%
.5%
.4%
.3%
For other Sections use = M/mbh²fc
m = 1.00 for Rectangular Sections. m = .65 for Octagonal Sections.

PLATE XXXV.

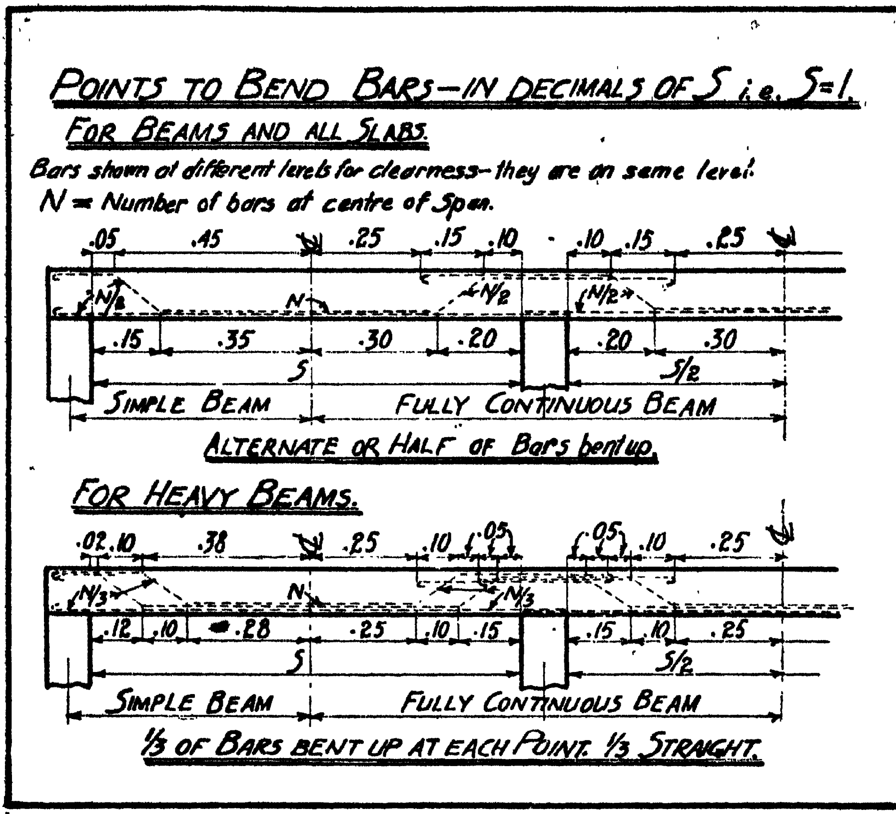

POINTS TO BEND BARS—IN DECIMALS OF S i.e. S=1.
FOR BEAMS AND ALL SLABS.
Bars shown at different levels for clearness—they are on same level.
N = Number of bars at centre of Span.
.05 .45 .25 .15 .10 .10 .15 .25
N/2 N N/2 N/2
.15 .35 .30 .20 .20 .30
S S/2
SIMPLE BEAM
FULLY CONTINUOUS BEAM
ALTERNATE OR HALF OF Bars bent up.
FOR HEAVY BEAMS.
.02 .10 .38 .25 .10 .05 .05 .10 .25
N/3 N N/3
.12 .10 .28 .25 .10 .15 .15 .10 .25
S S/2
SIMPLE BEAM
FULLY CONTINUOUS BEAM
1/3 OF BARS BENT UP AT EACH POINT. 1/3 STRAIGHT.

TABLE XXXVI.

Areas etc. of Round Bars and Wire.

Diameter. Ins.	Area. Sq. ins.	Circumference. Ins.	Weight Per ft. lbs.	Imperial. S.W.G.	Diameter. Ins.	Area. Sq. ins.	Circumference. Ins.	Weight. Per ft. lbs.
$\frac{1}{4}$	·049	·785	·168		$1\frac{1}{8}$	·994	3·534	3·380
					$1\frac{1}{4}$	1·227	3·927	4·172
$\frac{5}{16}$	·077	·982	·261					
$\frac{3}{8}$	·110	1·178	·376		$1\frac{3}{8}$	1·485	4·320	5·049
					$1\frac{1}{2}$	1·767	4·712	6·008
$\frac{7}{16}$	·150	1·375	·511					
$\frac{1}{2}$	·196	1·571	·668	20	·036	·00102	·113	·00346
				18	·048	·00180	·151	·00616
$\frac{9}{16}$	·249	1·767	·845					
$\frac{5}{8}$	·307	1·964	1·043	16	·064	·00322	·201	·0109
				14	·080	·00503	·252	·0171
$\frac{11}{16}$	·371	2·160	1·262					
$\frac{3}{4}$	·442	2·356	1·502	12	·104	·0085	·327	·0288
				10	·128	·0129	·403	·043[illegible]
$\frac{13}{16}$	·519	2·553	1·763					
$\frac{7}{8}$	·601	2·749	2·044	8	·160	·0201	·503	·068[illegible]
				6	·192	·0290	·604	·0985
$\frac{15}{16}$	·690	2·945	2·347					
1	·785	3·142	2·670					

TABLE XXXVII.

Spacing of Round Bars and Wire in Reinforced Slabs.

Diameter Inches.	Sectional area of Steel per foot Width of Slab when Spaced as follows :—													
	2″	2½″	3″	3½″	4″	4½″	5″	5½″	6″	7″	8″	9″	10″	12″
¼	·29	·23	·20	·17	·15	·13	·12	·11	·10	·08	·07	·07	·06	·05
5/16	·46	·36	·31	·26	·23	·20	·18	·17	·15	·13	·11	·10	·09	·08
⅜	·66	·53	·44	·38	·33	·29	·26	·24	·22	·19	·17	·15	·13	·11
7/16	·90	·72	·60	·51	·45	·40	·36	·33	·30	·26	·23	·20	·18	·15
½	1·18	·94	·78	·67	·59	·52	·47	·43	·39	·34	·29	·26	·24	·20
9/16	1·49	1·19	·99	·85	·75	·66	·60	·54	·50	·43	·37	·33	·30	·25
⅝	1·84	1·47	1·23	1·05	·92	·82	·74	·07	·61	·53	·46	·41	·37	·31
11/16	2·23	1·78	1·48	1·27	1·11	·99	·89	·81	·74	·64	·56	·49	·45	·37
¾	2·65	2·12	1·77	1·51	1·32	1·18	1·06	·96	·88	·76	·66	·59	·53	·44
13/16	3·11	2·48	2·07	1·78	1·56	1·38	1·24	1·13	1·04	·89	·78	·69	·62	·52
⅞	3·61	2·88	2·40	2·06	1·80	1·60	1·44	1·31	1·20	1·03	·90	·80	·72	·60
15/16	4·14	3·31	2·76	2·37	2·07	1·84	1·66	1·51	1·38	1·18	1·03	·92	·83	·69
1	4·71	3·77	3·14	2·69	2·36	2·09	1·88	1·71	1·57	1·35	1·18	1·05	·94	·78
1⅛	5·96	4·77	3·98	3·41	2·98	2·65	2·39	2·17	1·99	1·70	1·49	1·33	1·19	·99
1¼	7·36	5·89	4·91	4·21	3·68	3·27	2·95	2·68	2·45	2·10	1·84	1·64	1·47	1·23
1⅜	8·91	7·12	5·94	5·09	4·45	3·96	3·56	3·24	2·97	2·55	2·23	1·98	1·78	1·48
1½	10·60	8·48	7·07	6·06	5·30	4·71	4·24	3·86	3·53	3·03	2·65	2·36	2·12	1·77
S.W.G.														
20	·0061	·0049	·0041	·0035	·0031	·0027	·0024	·0022	·0020	·0018	·0015	·0014	·0012	·0010
18	·0108	·0087	·0072	·0062	·0054	·0048	·0043	·0039	·0036	·0031	·0027	·0024	·0022	·0018
16	·0193	·0155	·0129	·0111	·0097	·0083	·0077	·0070	·0064	·0055	·0048	·0043	·0039	·0032
14	·0302	·0242	·0201	·0172	·0151	·0134	·0121	·0111	·0105	·0087	·0075	·0067	·0060	·0050
12	·0510	·0408	·0340	·0293	·0255	·0226	·0204	·0185	·0170	·0146	·0127	·0113	·0102	·0085
10	·0775	·0620	·0515	·0442	·0387	·0344	·0310	·0281	·0258	·0218	·0193	·0172	·0155	·0129
8	·1210	·0965	·0805	·0690	·0604	·0536	·0483	·0440	·0402	·0346	·0302	·0268	·0241	·0201
6	·1740	·1390	·1160	·0995	·0870	·0775	·0696	·0633	·0580	·0500	·0435	·0387	·0348	·0290

Weight of Slab per Sq. Ft. lbs.	Moment of Resistance Ft. lbs.	Total Depth of Slab Inches.	For $M = \frac{wL^2}{8}$ 16′	18′	20′	Depth to centre of Steel Inches.	Depth below centre of Steel Inches.	Steel Area per Ft. width sq. in.	Spacing in inches of Round Bars for Diameters of ¼″	5/16″	⅜″	½″	⅝″	¾″	⅞″
25	147	2				1¼	¾	·101	5¾	9					
31	288	2½				1¾	¾	·141	4	6½					
38	476	3				2¼	¾	·181	3¼	5	7½				
44	712	3½				2¾	¾	·221	2½	4½	6				
50	995	4				3¼	¾	·261	2¼	3½	5	9			
56	1,160	4½				3½	1	·282	2	3¼	4½	8½			
63	1,510	5				4	1	·322		2¾	4	7½			
69	1,910	5½				4½	1	·362			3½	6½	10		
75	2,360	6				5	1	·402			3¼	5¾	9		
81	2,600	6½				5¼	1¼	·423			3	5½	8½		
88	3 110	7				5¾	1¼	·463			2¾	5	7¾	11½	
94	3,680	7½	115			6¼	1¼	·503			2½	4½	7½	10½	
100	4,300	8	135			6¾	1¼	·543			2¼	4½	6¾	9¾	
113	5,680	9	178	140		7¾	1¼	·623			2	3¾	5¾	8½	11½
125	7,230	10	226	179	145	8¾	1¼	·704				3¼	5	7½	10
138	8,500	11	266	210	170	9½	1½	·764				3	4¾	6¾	9½
150	10,400	12	326	257	208	10½	1½	·844				2¾	4½	6½	8½
Feet . . .			17·9	20·2	22·4	Spans for . . $M = \frac{wL^2}{10}$			Continuous Beams.						
Feet . . .			20·0	22·5	25·0	Spans for . . $M = \frac{wL^2}{12}$			Continuous Beams.						
Feet . . .			22·6	25·4	28·2	Spans for . . $M = \frac{wL^2}{16}$			Continuous Beams.						
Feet . . .			8·0	9·0	10·0	Spans for . . $M = \frac{wL^2}{2}$			Cantilevers.						

NOTES thin and a thicker one is desirable or the nearest Slab is too
rs should be increased as follows :—

able for slab used $\times \frac{\text{Load in Table}}{\text{Load to be designed for}}$

n, w = 90 lbs.

Limiting Load fo $\times \frac{162}{90} = 4''$.

PLATE XXXIX.

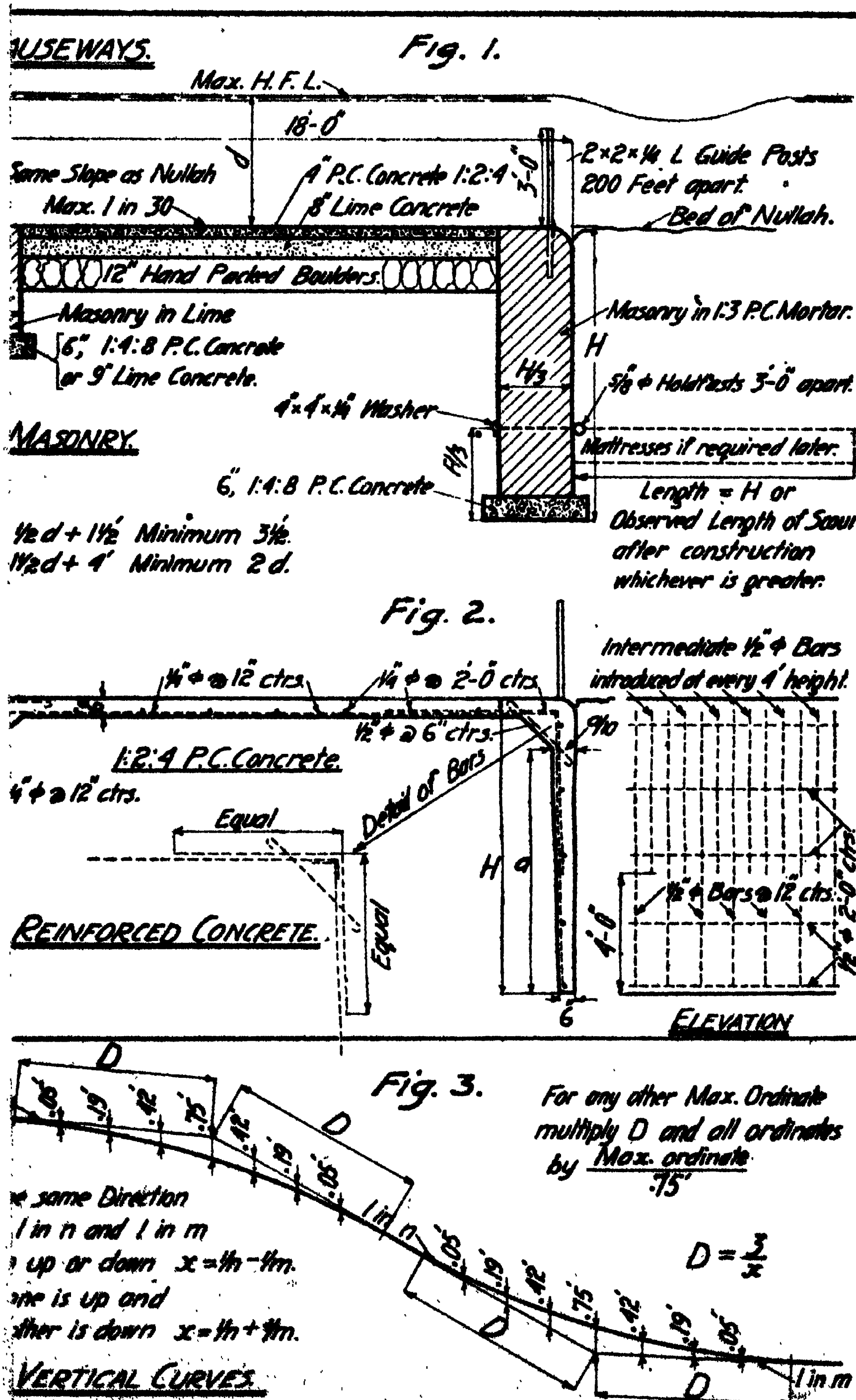
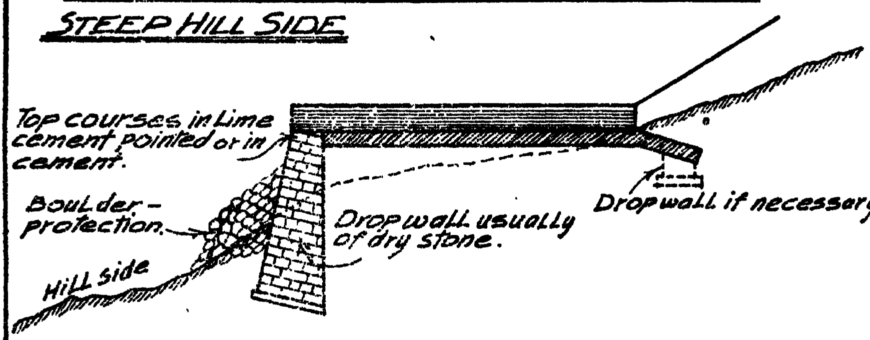

CAUSEWAYS.
Fig. 1.
Max. H.F.L.
18'-0"
d
Same Slope as Nullah
Max. 1 in 30
4" P.C. Concrete 1:2:4
8" Lime Concrete
3'-0"
2×2×¼ L Guide Posts
200 Feet apart
Bed of Nullah.
12" Hand Packed Boulders
Masonry in Lime
6" 1:4:8 P.C. Concrete
or 9" Lime Concrete.
Masonry in 1:3 P.C. Mortar.
H
H/3
5/8 φ Holdfasts 3'-0" apart.
4×4×¼ Washer
MASONRY.
H/3
Mattresses if required later.
6" 1:4:8 P.C. Concrete
Length = H or
Observed Length of Scour
after construction
whichever is greater.
½d + 1½' Minimum 3½'.
1½d + 4' Minimum 2d.
Fig. 2.
Intermediate ½" φ Bars
introduced at every 4' height.
¼" φ @ 12" ctrs.
¼" φ @ 2'-0" ctrs.
½" φ @ 6" ctrs.
9/10
1:2:4 P.C. Concrete.
¼" φ @ 12" ctrs.
Detail of Bars
Equal
Equal
H
a
4'-0"
½" φ Bars @ 12" ctrs.
½" φ @ 2'-0" ctrs.
REINFORCED CONCRETE.
6"
ELEVATION
D
Fig. 3.
For any other Max. Ordinate
multiply D and all ordinates
by Max. ordinate / .75
.05
.19
.42
.75
D
1 in n
same Direction
1 in n and 1 in m
up or down x = 1/n − 1/m.
one is up and
other is down x = 1/n + 1/m.
D = 3/x
1 in m
VERTICAL CURVES.

PLATE XL.

SCUPPER on a SLOPE

TRANSVERSE SECTION of SCUPPER on STEEP HILL SIDE



WIRE CRATING

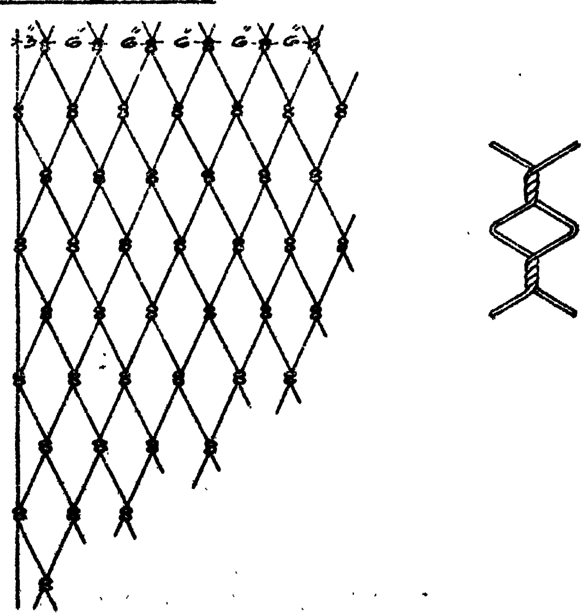
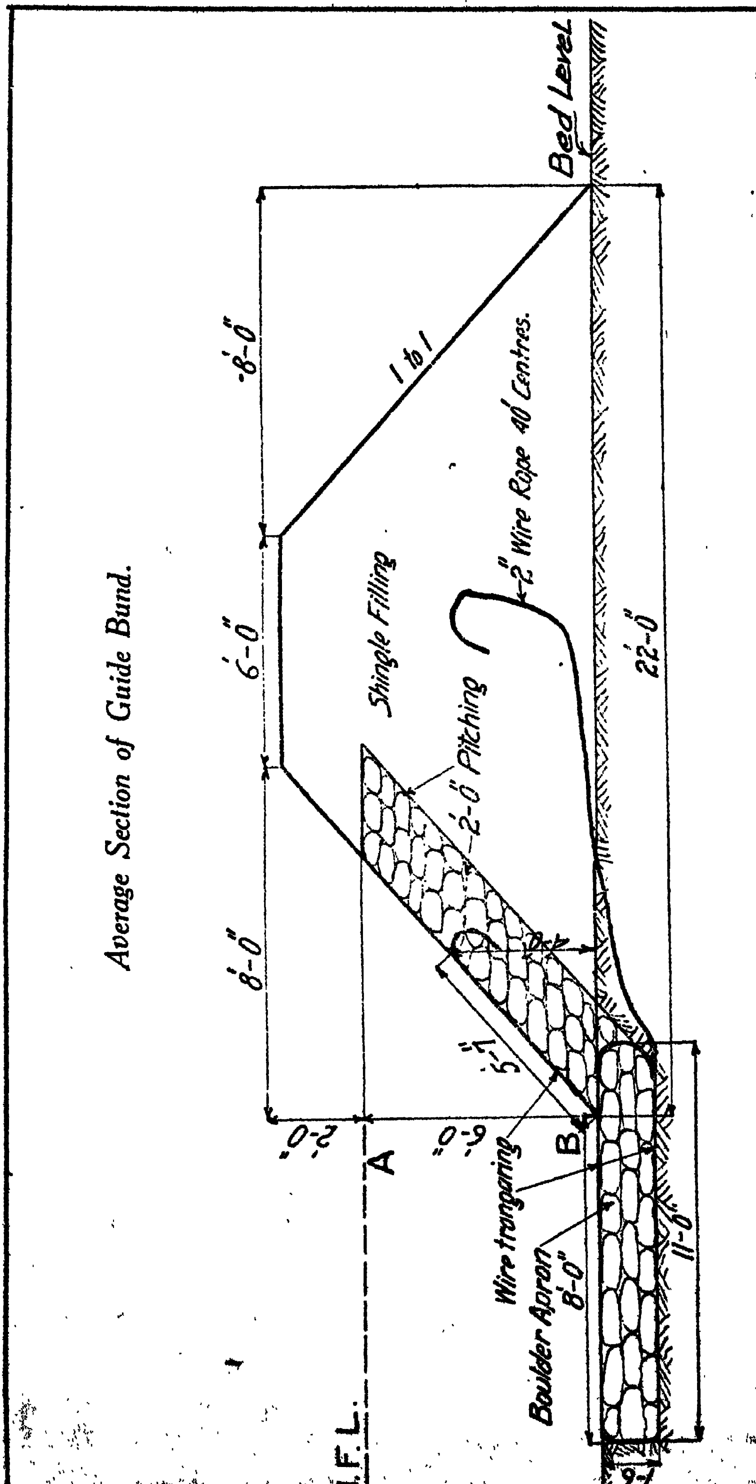


PLATE XLI.



Note.—The height A B, and resultant other dimensions will vary according to the height of high flood.

PLATE XLII.

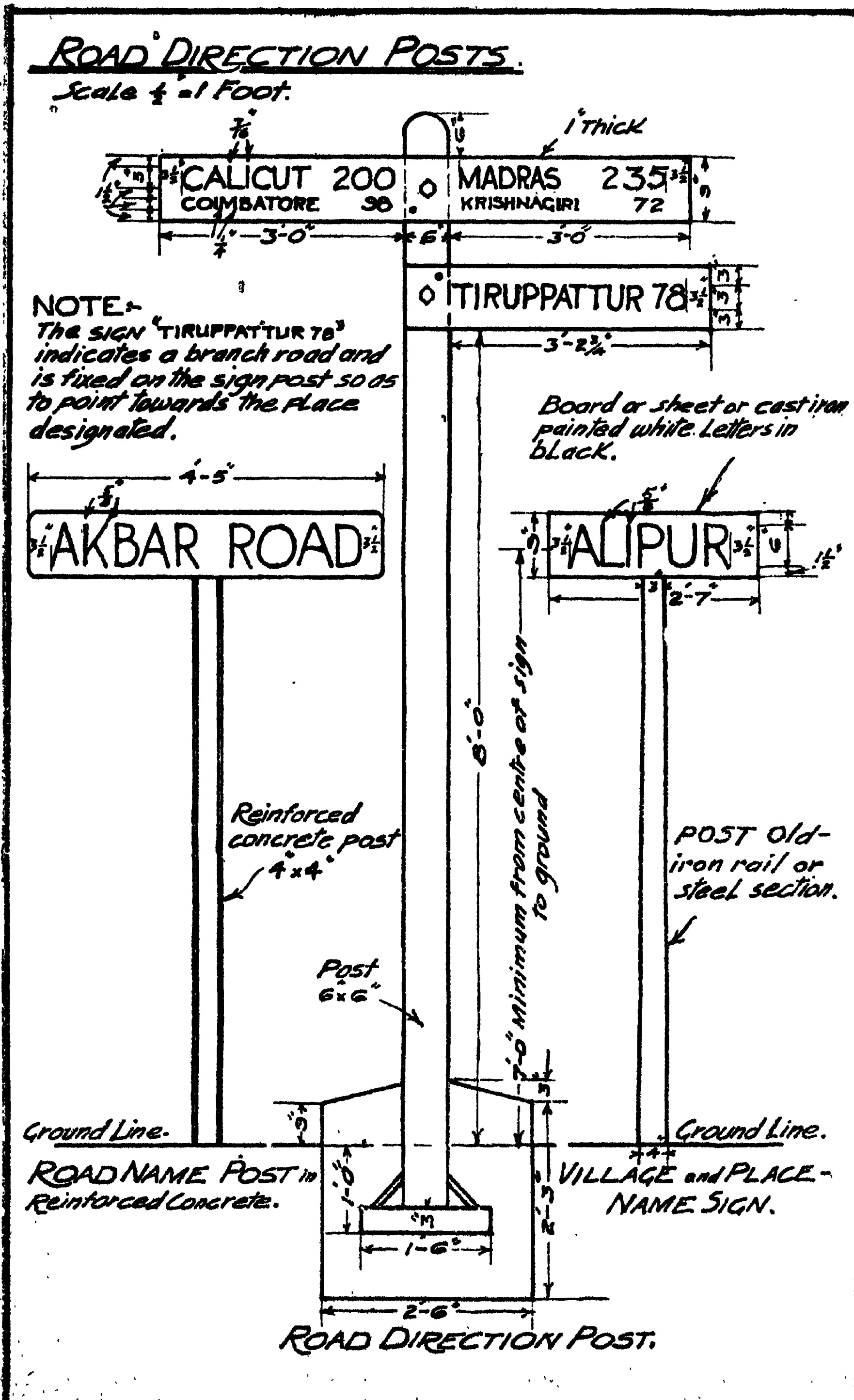

ROAD DIRECTION POSTS.
Scale 1/2" = 1 Foot.
1" Thick
CALICUT 200
COIMBATORE 98
MADRAS 235
KRISHNAGIRI 72
3'-0"
6"
3'-0"
TIRUPPATTUR 78
3'-2 3/4"
NOTE:-
The sign "TIRUPPATTUR 78" indicates a branch road and is fixed on the sign post so as to point towards the place designated.
Board or sheet or cast iron painted white. Letters in black.
4'-5"
AKBAR ROAD
ALIPUR
2'-7"
Reinforced concrete post 4"x4"
POST Old-iron rail or steel section.
8'-0"
7'-0" Minimum from centre of sign to ground
Post 6"x6"
Ground Line.
Ground Line.
ROAD NAME POST in Reinforced Concrete.
VILLAGE and PLACE-NAME SIGN.
1'-6"
2'-6"
ROAD DIRECTION POST.

PLATE XLIII.

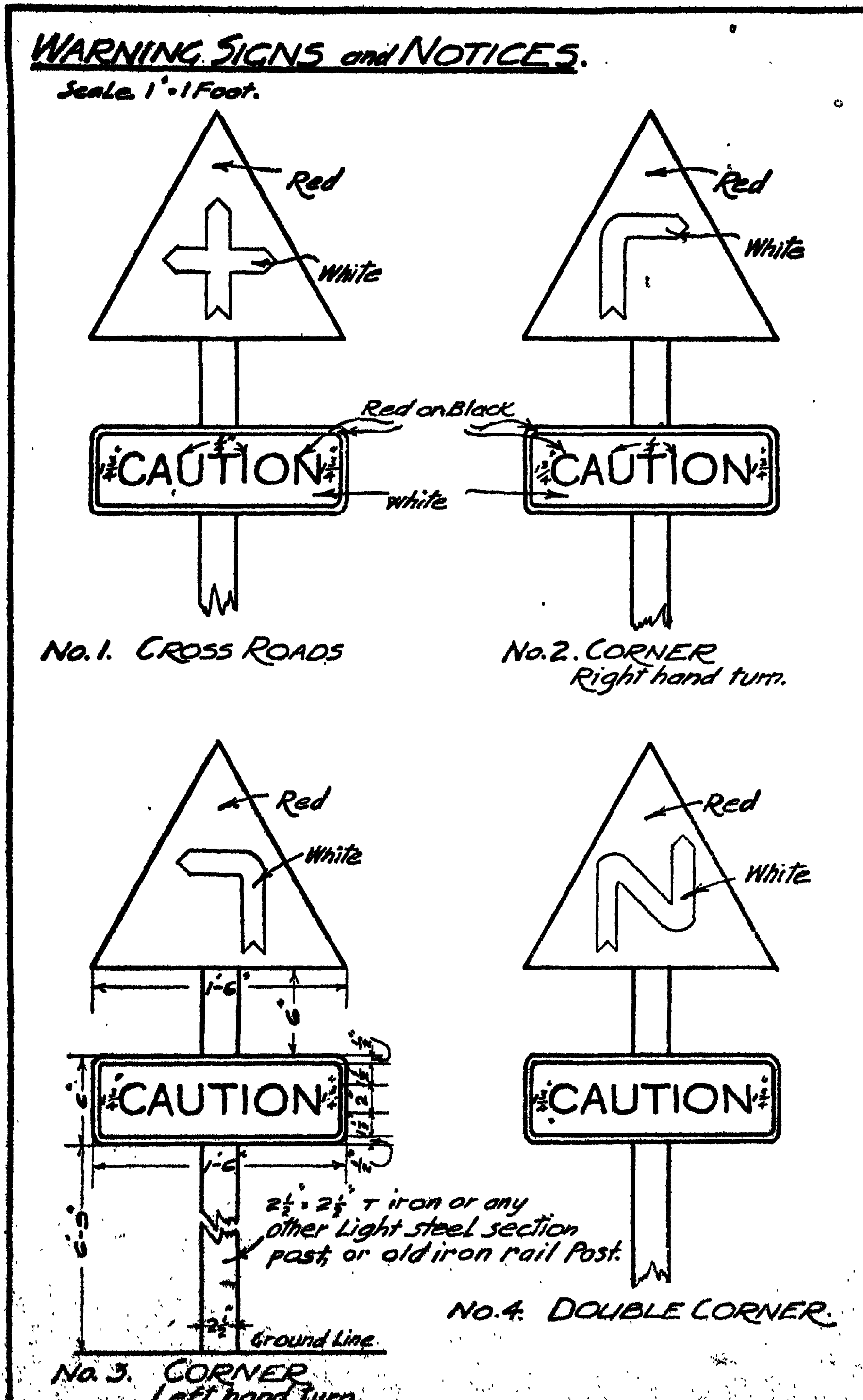

WARNING SIGNS and NOTICES.
Scale 1" = 1 Foot.
Red
White
Red on Black
CAUTION
white
No. 1. CROSS ROADS
No. 2. CORNER Right hand turn.
1'-6"
2½" × 2½" T iron or any other Light steel section post, or old iron rail Post.
Ground Line
No. 3. CORNER Left hand Turn.
No. 4. DOUBLE CORNER.

PLATE XLIV.

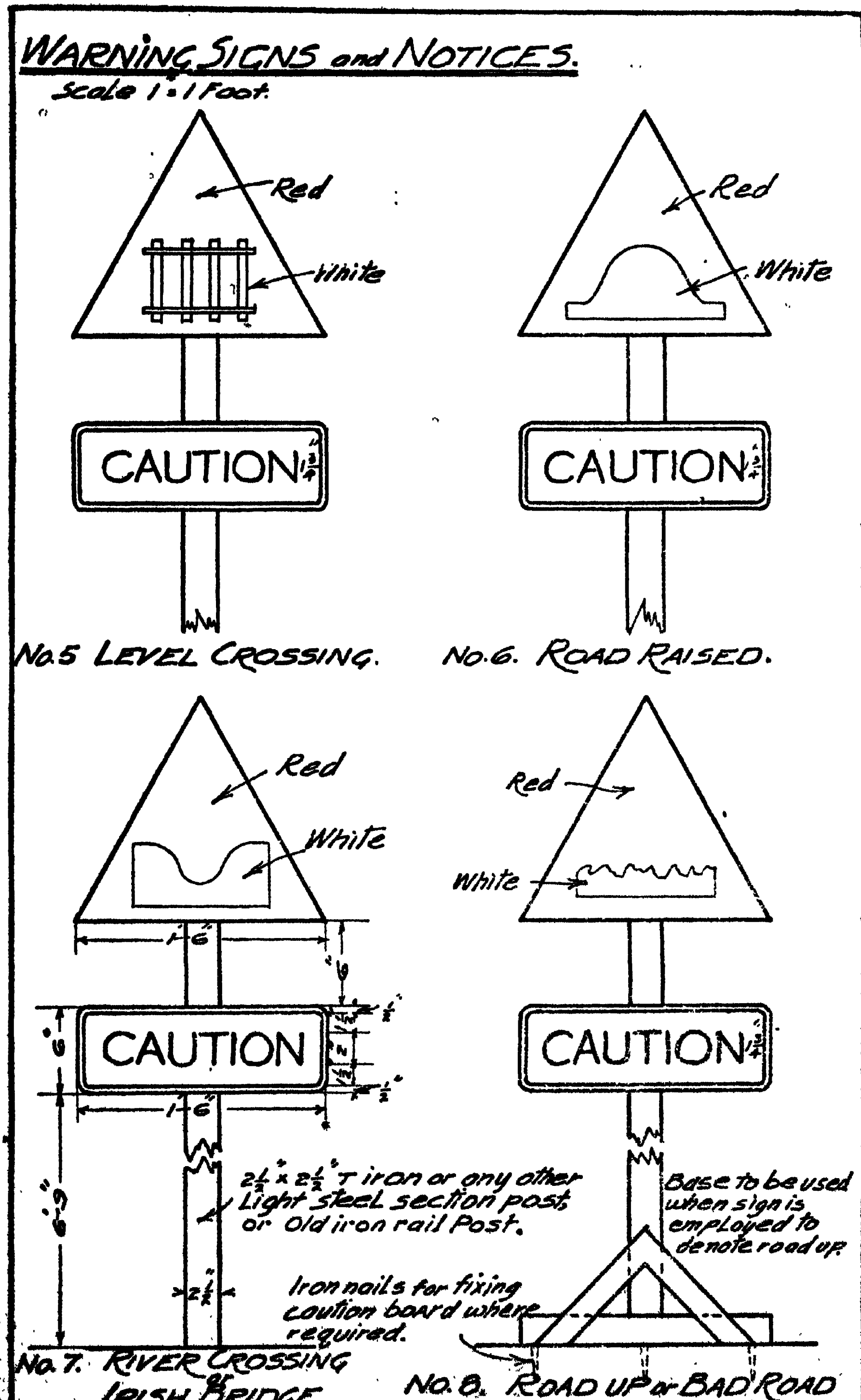

WARNING SIGNS and NOTICES.
Scale 1 = 1 Foot.
Red
White
CAUTION
No. 5 LEVEL CROSSING.
Red
White
CAUTION
No. 6. ROAD RAISED.
Red
White
1'-6"
6"
CAUTION
1'-6"
6'-9"
$2\frac{1}{2}$" × $2\frac{1}{2}$" T iron or any other Light steel section post, or Old iron rail Post.
$2\frac{1}{2}$"
Iron nails for fixing caution board where required.
No. 7. RIVER CROSSING or IRISH BRIDGE
Red
White
CAUTION
Base to be used when sign is employed to denote road up.
No. 8. ROAD UP or BAD ROAD

PLATE XLV.

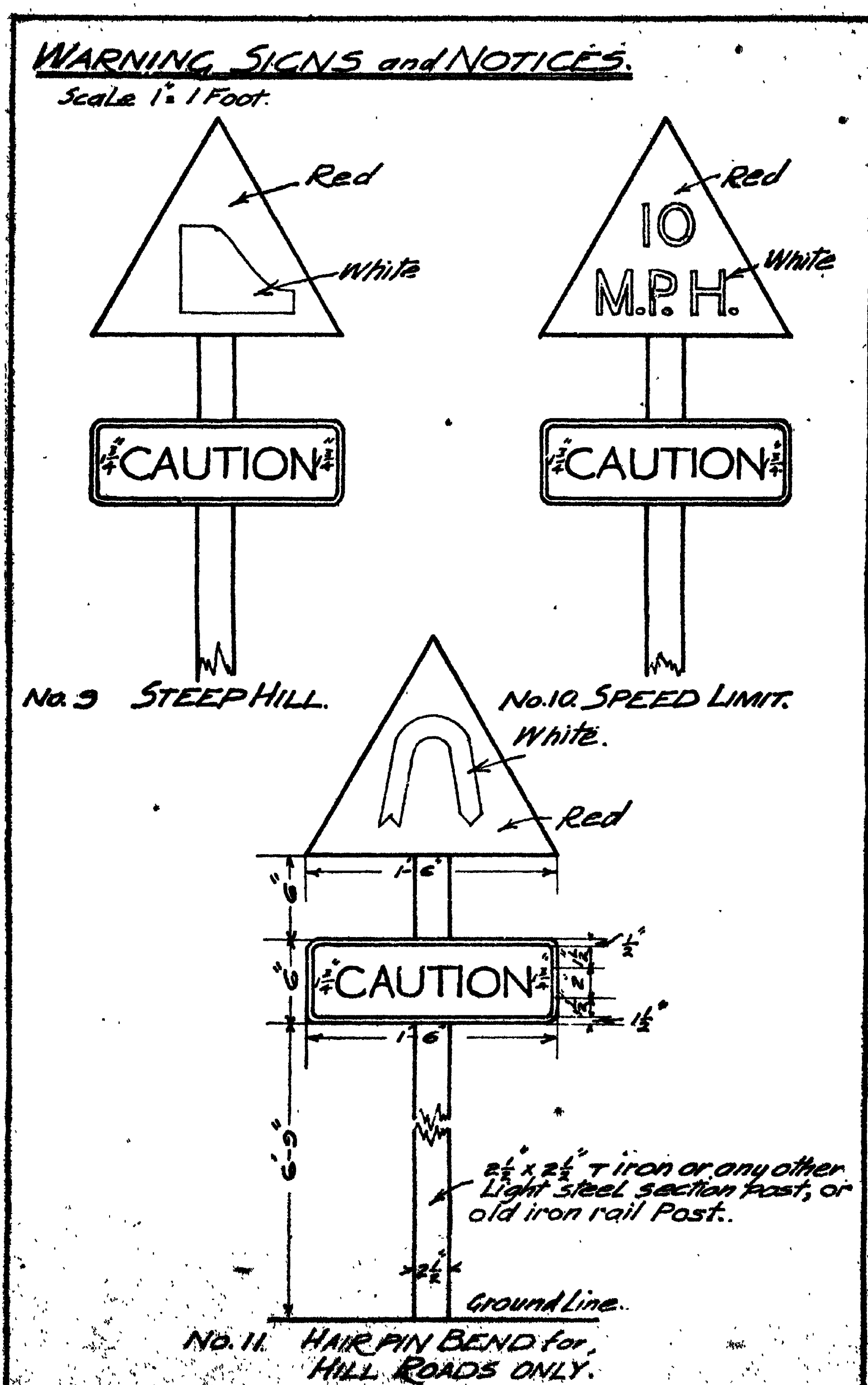

WARNING SIGNS and NOTICES.
Scale 1" = 1 Foot.
Red
White
CAUTION
No. 9 STEEP HILL.
Red
10
M.P.H.
White
CAUTION
No. 10 SPEED LIMIT.
White.
Red
1'-6"
6"
CAUTION
6"
½"
1½"
1'-6"
6'-0"
2½" x 2½" T iron or any other Light steel section post, or old iron rail Post.
2½"
Ground Line.
No. 11. HAIR PIN BEND for HILL ROADS ONLY.

PLATE XLVI.

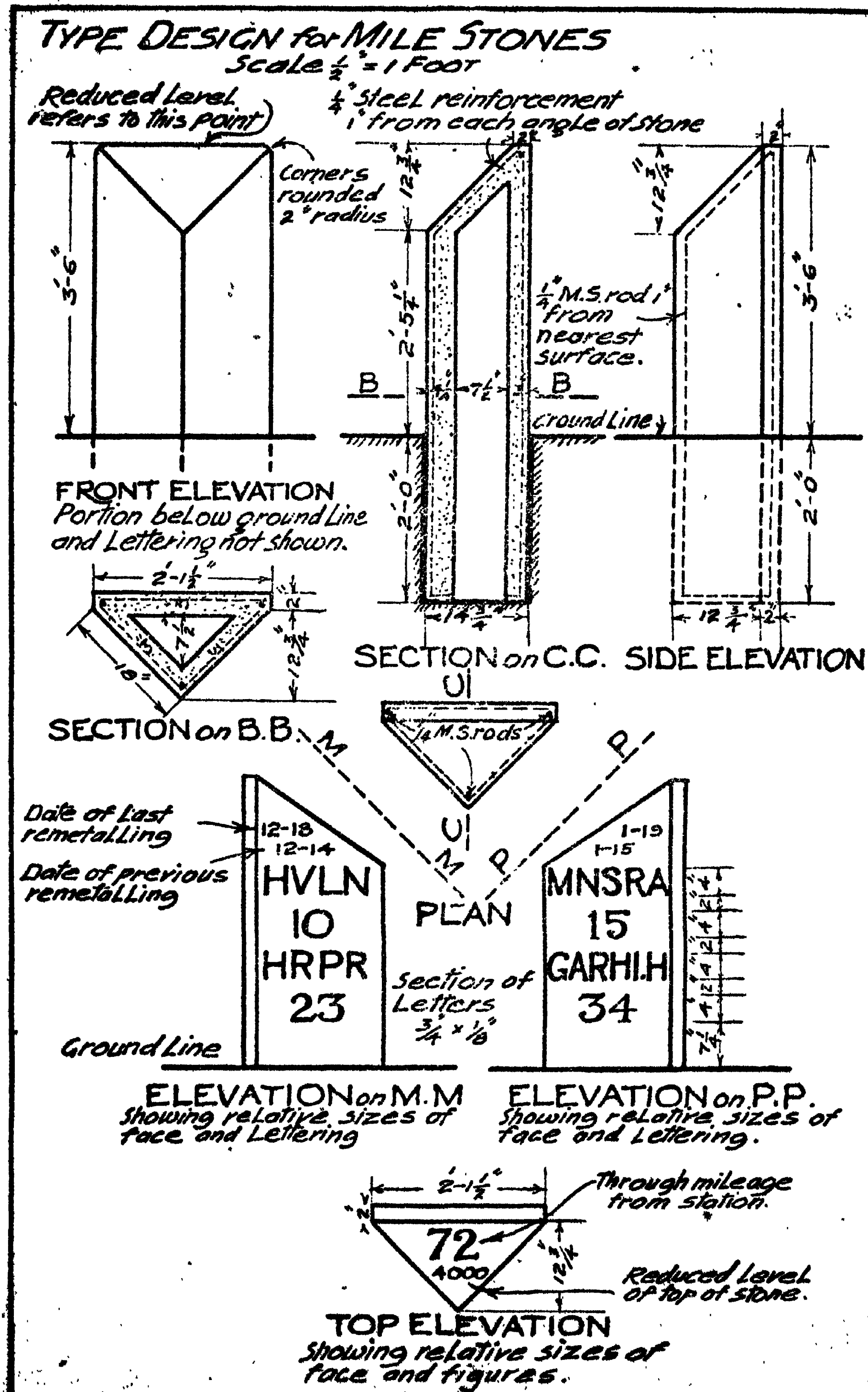
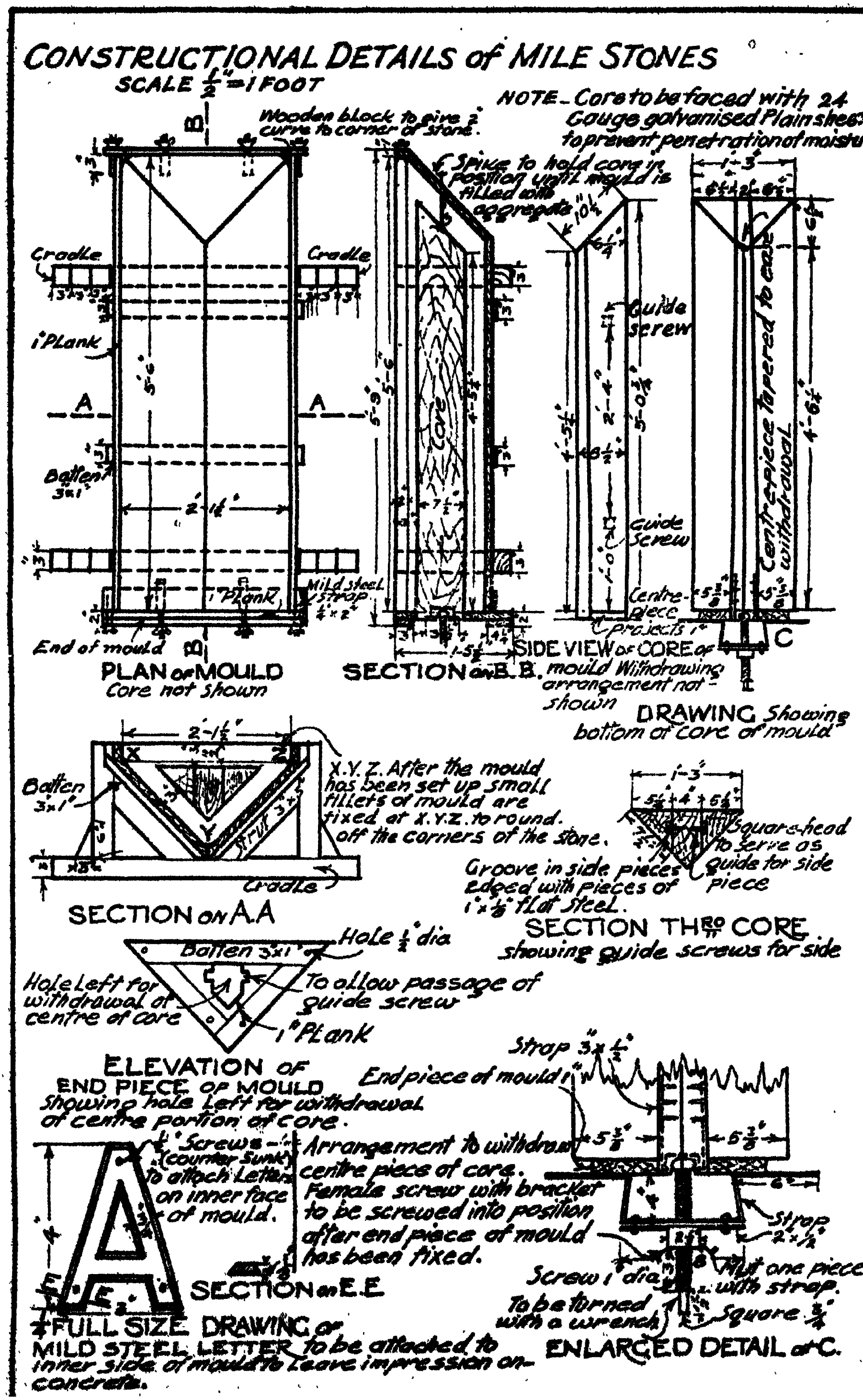
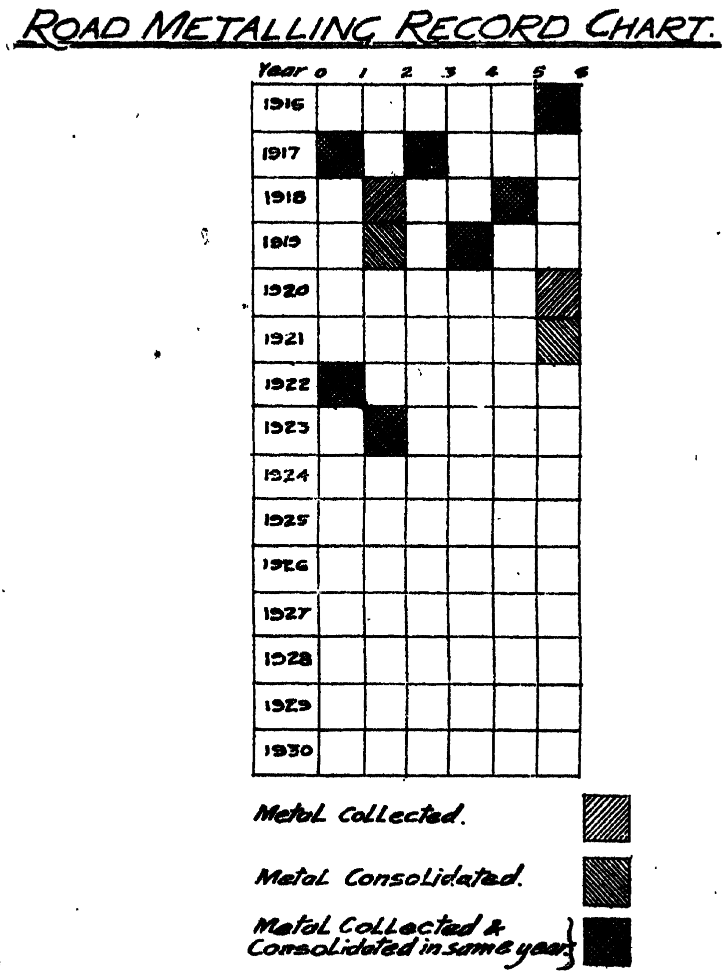

TYPE DESIGN for MILE STONES
Scale ½" = 1 Foot
Reduced Level refers to this point
¼" Steel reinforcement 1" from each angle of Stone
Corners rounded 2" radius
¼" M.S. rod 1" from nearest surface.
Ground Line
FRONT ELEVATION
Portion below ground Line and Lettering not shown.
SECTION on B.B.
SECTION on C.C.
SIDE ELEVATION
¼" M.S. rods
PLAN
Date of Last remetalling
Date of previous remetalling
12-18
12-14
HVLN
10
HRPR
23
1-19
1-15
MNSRA
15
GARHI.H
34
Section of Letters ¾" × ⅛"
ELEVATION on M.M
Showing relative sizes of face and Lettering
ELEVATION on P.P.
Showing relative sizes of face and Lettering.
Through mileage from station.
72
4000
Reduced Level of top of stone.
TOP ELEVATION
Showing relative sizes of face and figures.

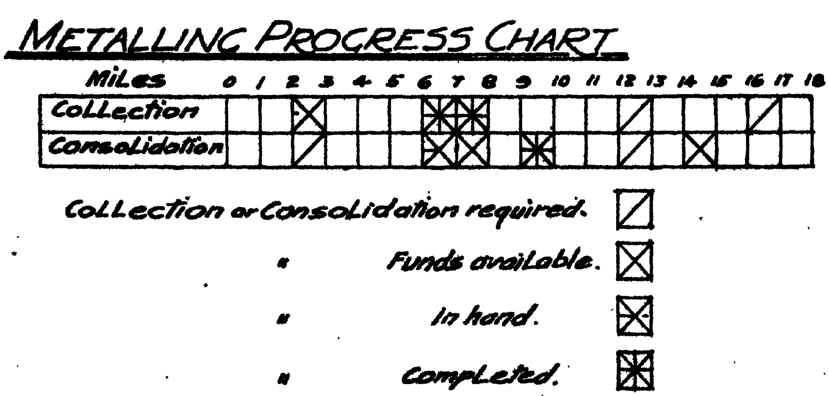
PLATE XLVII.



CONSTRUCTIONAL DETAILS of MILE STONES
SCALE ½"=1 FOOT
NOTE – Core to be faced with 24 Gauge galvanised Plain sheet to prevent penetration of moisture.
Wooden block to give 2" curve to corner of stone.
Spike to hold core in position until mould is filled with aggregate
Cradle
1" Plank
Batten 3×1"
Mild steel strap ¼×2"
End of mould
PLAN of MOULD
Core not shown
SECTION on B.B.
Guide screw
Centre-piece
Centre-piece tapered to ease withdrawal
C projects 1"
SIDE VIEW of CORE of mould Withdrawing arrangement not shown
DRAWING Showing bottom of core of mould
X.Y.Z. After the mould has been set up small fillets of mould are fixed at X.Y.Z. to round off the corners of the stone.
Strut 3×1"
SECTION on A.A
Square-head to serve as guide for side piece
Groove in side pieces edged with pieces of 1"×⅛" flat steel.
SECTION THRO CORE.
showing guide screws for side
Hole ½" dia
Batten 3×1"
Hole Left for withdrawal of centre of core
To allow passage of guide screw
1" Plank
ELEVATION OF END PIECE OF MOULD
Showing hole Left for withdrawal of centre portion of core.
Strap 3×¼"
End piece of mould 1"
¼" Screws – (counter Sunk) to attach Letters on inner face of mould.
Arrangement to withdraw centre piece of core.
Female screw with bracket to be screwed into position after end piece of mould has been fixed.
SECTION on E.E
Strap 2×½"
Nut one piece with strap.
Screw 1" dia
To be turned with a wrench
Square ¾"
¼ FULL SIZE DRAWING of MILD STEEL LETTER to be attached to inner side of mould to leave impression on concrete.
ENLARGED DETAIL of C.

PLATE XLVIII.





APPENDIX XLIX.

British road board specifications for the tar treatment of roads.

Road Board Specification No. 1.

GENERAL DIRECTIONS FOR SURFACE TARRING ON WATER-BOUND ROAD.

1. Surface tarring may be advantageously applied either to an old road surface in good condition or to a new surface after it has been consolidated and dried, but the tarring should never be carried out unless the road is thoroughly dry.

If there are any depressions, pot-holes, waves, grooves, or other irregularities, these should as far as practicable be made good before tarring is commenced, so as to provide an even surface.

2. Painting and spraying machines get through the work of tarring more rapidly than application by hand, and consequently are to be recommended, but hand work gives satisfactory results, and the selection of the method to be employed must be largely determined by the available supply of efficient labour.

3. If it is intended to tar an old surface it is advisable to take advantage of the early months of the year to scrape or brush the road during wet weather as a preparation for subsequent tarring, and especially to keep the road free from caked mud.

4. If the crust of a road is thin at the sides, but adequate in the centre, the sides should be strengthened and consolidated before application of tar to the surface.

5. In re-surfacing any road the surface of which is afterwards to be tarred, stone chippings, and not fine material, should be used for binding.

6. The road whilst being tarred should be closed to traffic over half its width, or, where practicable, over its whole width.

7. The road should be thoroughly brushed and cleaned before application of the tar. Wet brushing should be used some time previous to dry brushing, if there is any caked mud. Any method of brushing may be used which will scour and clean the road thoroughly, the best being horse brushing, followed by hand brushing.

8. Tar should be used for surface tarring which complies with either Road Board Specification for Tar No. 1 or Road Board Specification for Tar No. 2, but if the heavier grade of the tar is used, care should be taken to apply it only when the road is well warmed by the sun's rays, otherwise it will not flow freely.

9. The tar should be heated to its boiling point at convenient positions on the works, and should be applied as hot as possible, so that it may flow freely. The desired temperature will be generally found in practice to lie between 220° and 240° Fahrenheit for Tar No. 1 and between 260° and 280° Fahrenheit for Tar No. 2.

10. In order that the tar should be applied to the road as hot as possible, it is advisable if the method of application is by hand, to use flexible pipes to convey the tar from the boiler to the point of application. If these are not available, it will be found convenient, in case of hand pouring, to use 3-gallon cans specially constructed for the purpose, fitted with spouts leading direct from the bottom of the cans, and being not less than 1½ inches in diameter at the orifice.

11. Immediately on application the liquid tar should be brushed so far as necessary to ensure regularity in thickness of the coating.

12. The quantity of tar required will vary according to the physical conditions of the road, but generally in the case of a road to be treated with tar for the first time, the quantity should be one gallon to coat from five to seven superficial yards.

13. If the road must be opened for traffic before the tar has set hard, grit should be spread on the surface to prevent the tar from adhering to the wheels of vehicles, but gritting should be delayed as long as possible and the quantity of gritting material to be spread should be no more than sufficient to prevent the tar from adhering to wheels. Stone chippings, crushed gravel, coarse sand, or other approved material (free from dust) not larger than will pass through a ¼-inch square mesh should be used for gritting.

14. Precaution should be taken to prevent liquid tar passing directly through drainage gratings or outlets.

15. For the safety of the public precautions should be taken by lighting, watching, and warning.

Notice boards should be placed in suitable positions bearing in large letters printed in conspicuous colours the following words:—

CAUTION.

TARRING IN PROGRESS.

CYCLISTS ADVISED TO WALK.

It is specially desirable to place warning notices at points in the neighbourhood of the work where other roads join or cross the road being tarred, to enable motorists and cyclists to avoid the obstructed road by taking any available alternative route.

16. On heavily trafficked road it is advisable to apply a second coat to either the whole width or from 9 to 12 feet of the centre of the road in quantity of one gallon to coat from 8 to 10 yards super, about two to three months after the first application.

17. Surface tarring should be renewed annually on all important roads, and as required on roads with light traffic. On such re-coatings the quantity of tar to be applied will vary with the extent to which the previous coating of tar has been removed by weather or by traffic.

18. Two or more samples of the tar used should in all cases be kept in quart tin cans, and be carefully labelled, including particulars fixing the locality or length of the road on which the tar was used. The Road Board will arrange with the National Physical Laboratory to submit a selection of these samples to a series of chemical and physical tests with a view to the results being recorded for future reference, and surveyors will from time to time be invited to send samples for the purpose.

19. In all cases careful record should be kept of the condition of the road surfaces in winter and summer, both before and after tarring, the quantity and quality of tar used, the superficial area covered, the state of the weather when the work is being done, the time occupied in actual work, and in waiting whilst work is stopped owing to wet weather, the number of men employed and full details of the cost of labour and material.

20. Surveyors are recommended to have samples of the tar supplied to them under contracts properly tested by a qualified analytical chemist for—

(1) Specific gravity.
(2) Freedom from water.

(3) Fractionation.

(4) Free carbon.

NOTE.—These general directions are not intended to displace or to discourage the use of proprietary articles of which there are several of proved value.

Road Board Specification No. 2.

GENERAL DIRECTIONS FOR SURFACING WITH TAR MACADAM.

1. Any road which is to be surfaced with tar macadam should have a proper foundation or sub-crust of adequate thickness to bear the traffic likely to use it.

2. Before laying a new surface of tar macadam the thickness of the old crust, including foundation, should be ascertained by opening trial trenches at intervals averaging about 150 yards extending from the haunch of the road to the centre, such trenches to be made alternately on opposite sides of the road.

3. The thickness of the surface coating of tar macadam when consolidated by rolling should be not less than three inches, according to traffic requirements. For a greater thickness than 3 inches the material should be applied in two coats.

4. In the case of naturally hard sub-soils, not materially softened by infiltration of surface water, the total thickness of the road crust, including foundation, if any, after consolidation of the new surface of tar macadam by rolling, should not under ordinary circumstances be less than 6 inches, unless the sub-soil is so hard as in itself to act as a good foundation, in which case the thickness of the road crust may be reduced to 4 inches. In the case of clay or other yielding sub-soils the total thickness should not be less than 11 inches.

5. The finished surface should have a cross fall of about 1 in 32.

If the crust is not sufficiently thick at the crown to enable this cross fall to be obtained with a new coating of the thickness above mentioned, then the old surface should be left intact and unscarified and the thickness of the new coat of tar macadam increased as far as may be necessary.

If the crust is of sufficient thickness for the purpose, the regulation of the cross fall should be carried out by scarifying the surface and removing material from the crown to the sides previous to the application of the new coating. The material loosened by scarifying should be screened and all finer material than ½ inch should be thrown aside.

6. The aggregate of the new surface of tar macadam should be composed of broken stone of approved quality, or selected slag of approved quality, and should consist of 60 per cent of 2-inch Standard gauge, and 30 per cent of 1½-inch Standard gauge, and 10 per cent of ¾-inch to ½-inch size should be used for filling the voids during rolling operations.

In the case of two-coated work the sub-crust should consist of 2-inch Standard gauge stone, and the wearing suface 1½-inch Standard gauge stone; 10 per cent of ¾-inch to ½-inch size stone should be used for filling the voids during rolling operations.

7. The stone used must be thoroughly dried before being coated with tar.

8. For making tar macadam tar should be used which complies with Road Board Specification Tar No. 1, or Road Board Specification Tar No. 2, the choice being determined by the circumstances of each case.

If tar No. 1 has been used for tarring the stone, care should be taken, especially in hot weather, that the tarred material has been allowed to stand

for a sufficient length of time to allow the tarred surface of the stones to become partially hardened and in a tacky condition.

If tar No. 2 has been used for tarring the stone, the macadam should be laid soon after being tarred, and the stone coated with such tar should preferably be laid when the road is quite dry and in warm sunny weather.

9. The quantity of tar used to coat one ton of stone should be approximately from 9 to 12 gallons, varying according to the sizes of the stone, the grade of tar used, the method of mixing and other conditions.

10. The tar macadam after having been spread and levelled should be rolled into a smooth surface, but too much rolling should be avoided.

Less rolling is required than in the case of water-bound macadam.

A 10-ton roller is a suitable size for use in most cases, but good results can be obtained by using an 8-ton roller and finishing with a 10-ton roller.

11. In order to get the best results from the use of tar macadam it is advisable to apply a coating of tar to the surface after the road has been used by traffic for several weeks, not less than one gallon of tar being used for every 6 super yards of road surface. This tar should comply with the provisions of Road Board Specification for Tar No. 2 and should be poured or sprayed on the surface at a temperature of about 270° Fahrenheit.

12. Stone chippings, crushed gravel, coarse sand or other approved material (free from dust) not larger than will pass through a ½-inch square mesh should be used for gritting.

NOTE.—These general directions are not intended to displace or discourage the use of proprietary articles of which there are several of proved value.

Road Board Specification No. 3.

GENERAL DIRECTIONS FOR SURFACING WITH PITCH-GROUTED MACADAM.

1. Any road which is to be surfaced with pitch-grouted macadam should have a proper foundation or sub-crust of adequate thickness to bear the traffic likely to use it.

2. Before laying a new surface of pitch-grouted macadam the thickness of the old crust, including foundations, should be ascertained by opening trial trenches at intervals averaging about 150 yards extending from the haunch of the road to the centre, such trenches to be made alternately on opposite sides of the road.

3. The thickness of the surface coating of pitch-grouted macadam when finished and consolidated by rolling should be 2½ inches to 3 inches (except on very light traffic roads, when the thickness may be 2 inches) for single pitch-grouting, and from 4 inches to 4½ inches for the double pitch-grouting hereafter described.

4. In the case of naturally hard sub-soils, not materially softened by infiltration of surface water, the total thickness of the road crust, including foundation, if any, after consolidation by rolling of the new pitch-grouted surface, should not under ordinary circumstances be less than 6 inches, unless the sub-soil is so hard as in itself to act as a good foundation, in which case the thickness of the road crust may be reduced to 4 inches. In the case of clay or other yielding sub-soils the total thickness should not be less than 11 inches.

5. The finished surface should have a cross fall of about 1 in 32.

If the crust is not sufficiently thick at the crown to enable this cross fall to be obtained with a new coating of the thickness above mentioned, then the old surface should be left intact and unscarified, and the thickness of the new pitch-grouted coating increased as far as may be necessary.

If the crust is of sufficient thickness for the purpose, the regulation of the cross fall should be carried out by scarifying the surface and removing material from the crown to the sides previous to the application of the new coating. Material loosened by scarifying should be screened and all material finer than half-inch should be thrown aside.

6. The material to form the pitch-grouted macadam should be of approved quality broken to Standard 1½-inch gauge. In addition to this, 10 per cent of chipping of the same stone, varying from ¾ inch down to ⅜ inch, should be used for ⅜ closing after the grouting with the melted pitch.

7. For making pitch-grouted macadam the pitch used should comply with the Road Board Specification for Pitch, its viscosity being altered to suit climatic and local conditions by varying the quantity of the tar oils as specified therein.

8. It is important that the pitch should not be poured if the surface of the stone is wet. The stone may be protected by tarpaulins, or, if wet, may be dried *in situ* by portable blowers or other means.

9. The material after having been spread and levelled must be rolled down dry without chippings until the stones present a mosaic surface.

10. The quantity of pitch required to grout a single coating is approximately for a consolidated thickness of 2 inches 1¼ gallons per yard super, for 2½ inches 1½ gallons per yard super, and for 3 inches 2 gallons per yard super, but these quantities may vary with different materials, and care must always be taken to fill the voids.

11. The pitch, after being carefully melted as described in Clause 18, must be raised to a temperature of 300° Fahrenheit. Clean sharp sand must be heated on sand heaters to a temperature of 400° Fahrenheit. A dandy, or portable mixing vessel, is then to be filled with equal parts, by measurement, of the heated pitch and the hot sand and the mixture, hereafter called the matrix, is to be kept well stirred while it is being emptied from the dandy, or portable mixing vessel, into pouring cans of from 2 to 3 gallons capacity which are used for pouring the matrix on to the roadway. Not only during the process of mixing but afterwards right up to the time of actual pouring, the matrix must be kept well stirred. The matrix prepared with pitch in the quantities specified in Clause 10 should be sufficient to fill the voids.

12. The final rolling should be commenced immediately after pouring the pitch matrix, and carried on rapidly before the matrix has time to set. The 10 per cent of graded chippings should be spread over the grouted surface in part previously to and the remainder during the process of rolling. The traffic may be allowed on to the finished surface as soon as the surface has cooled to the normal temperature.

Double pitch-grouting.

13. When the traffic is so heavy that a consolidated thickness of from 4 inches to 4½ inches of pitch-grouted macadam is required, it is desirable, in order to obtain the best and most economical results, to divide the coating into two layers, the bottom layer to be the thicker one and to consist of large stone, the two layers being rolled down and grouted separately. Any local stone which can be procured cheaply may, if suitable in quality for foundation work, be used for the bottom layer graded from 3-inch Standard gauge down to 2-inch Standard gauge. No chippings are required for finishing the rolling of the bottom layer. The material for the upper layer should consist of hard road stone of approved wearing quality, broken to 1½-inch Standard gauge, and 10 per cent of chippings of the same stone used for the upper layer, graded from half-inch down to quarter inch, should be added

before and during the process of rolling, and rolled down so as to form the finished surface of the road.

14. In pouring the pitch on the bottom layer the surface of the pitch should not be brought to the surface of the stone, but should lie about half-inch below such surface, with the object of providing a key for the upper layer.

15. The materials and the methods of grouting and laying down in the case of double pitch-grouting should, except when otherwise expressly stated, conform to the provisions of Clauses 7, 8, 9, 11 and 12.

16. The quantity of pitch required for double pitch-grouting is approximately for a consolidated thickness of 4 inches 3¼ gallons per yard super, and for 4½ inches 3½ gallons per yard super, but these quantities may vary with different materials, and care must always be taken to fill the voids in the surface coating adequately.

17. For the purpose of accurately ascertaining the proportions necessary for the matrix, it is essential that portable weights, scales and measures be provided, and all materials used in the preparation of the matrix should be accurately proportioned by weight or measurement.

INSTRUCTIONS FOR MELTING THE PITCH.

18. The pitch boilers of from two or three tons capacity should be charged with pitch and about one-half of the proper proportion of tar oils. The fire should then be lighted, and thereafter a steady fire, with fire-doors closed, should be maintained, when, in from four to five hours, the pitch should be thoroughly melted. A bright fire should be kept until the pitch reaches a temperature of 300° Fahrenheit, when the remainder of the oils should be added and the mixture thoroughly stirred; the fire-doors should then be opened and the temperature of the melted pitch permitted to fall to 250° or 270° Fahrenheit. The pitch should then be ready for use, and in all cases should be thoroughly well stirred before being drawn off.

In the event of bad weather stopping the work of grouting the fire-door should be left open, the damper closed, and the temperature of the pitch allowed to fall to 200° Fahrenheit. It can be kept at this temperature for long periods with banked fires consuming about 7 lbs. of coke per hour.

It is recommended that a suitable Fahrenheit thermometer with metal protection should be at hand to indicate the temperature of the melted pitch. Whenever the weather is favourable for the recommencement of the work the pitch must be again raised to 270° Fahrenheit by closing the doors and sharp firing.

It is desirable that the boiler should be kept air-tight when the pitch is being melted, by the use of air-tight covers properly packed so as to make an air-tight joint.

NOTE.—These general directions are not intended to displace or to discourage the use of proprietary articles of which there are several of proved value.

Road Board Specification No. 4.

SPECIFICATION FOR TAR NO. 1.

1. This tar is suitable for the surface tarring of roads. General.

As to the use of this tar for making tar macadam, see "Road Board General Directions for Surfacing with Tar Macadam."

2. The tar should be heated to such a temperature that it will reach the road surface in a highly fluid condition. The necessary temperature to attain this end will vary with the mode of application of the tar. The tar should Heating.

be heated in a heater or "boiler" specially designed to prevent frothing, which will otherwise inevitably occur if the tar contains even a small percentage of water. The desired temperature will generally be found in practice to be between 220° and 240° Fahrenheit or 104° and 116° Centigrade in the heater or boiler.

Source of the tar.

3. The tar shall be derived wholly from the carbonization of bituminous coal except that it may contain not more than 10 per cent of its volume of the tar (or distillates or pitch therefrom) produced in the manufacture of carburetted water gas.

Specific gravity.

4. The specific gravity of the tar at 15° Centigrade (59° Fahrenheit) shall be as nearly as possible 1·19, and in no case shall it be lower than 1·16 or higher than 1·22.

Freedom from water and ammon'a.

5. The tar shall be commercially free from water, *i.e.*, it shall not contain more than 1 per cent by volume of water or ammoniacal liquor, which water or liquor (if present) shall not contain more ammonia, free or combined, than corresponds to five grains of ammonia per gallon (=70 milligrammes per litre) of the tar.

The amount of water or liquor is to be determined by condensation from the products of distillation of the tar by cooling with a cold water condenser. Any water so condensed, after measurement, should be separated from light oils which may have condensed with it, and the amount of ammonia in it should be estimated by direct titration with Standard acid. The amount of ammonia thus determined should be calculated in terms of grains of ammonia per gallon of tar.

Fractionation.

6. On distillation in a litre fractionating flask one-half to two-thirds filled, the tar shall yield the proportions by weight of distillates stated below; the temperatures of distillation being read on a thermometer of which the bulb is opposite the side tube of the flask—

Below 170° Centigrade or 338° Fahrenheit, not more than 1 per cent of distillate (light oils), exclusive of water.

Between 170° and 270° Centigrade or 338° and 518° Fahrenheit, not less than 16 per cent and not more than 26 per cent of distillate (middle oils).

Between 270° and 300° Centigrade or 518° and 572° Fahrenheit, not less than 3 per cent and not more than 10 per cent of distillate (heavy oils).

The total distillate between 170° and 300° Centigrade, or 338° and 572° Fahrenheit, shall be not less than 24 per cent and not more than 34 per cent, *i.e.*, where the middle oils approach the maximum allowed, the heavy oils should approach the minimum allowed and *vice versâ*.

Naphthalene.

7. The distillate between 170° and 270° Centigrade, or 338° and 518° Fahrenheit (middle oils), shall remain clear and free from solid matter (crystals of naphthalene, etc.) when maintained at a temperature of 30° Centigrade for half an hour.

This requirement may be waived in the case of tar supplied direct from gas works, but tar from which the naphthalene has been extracted is preferable to tar containing much naphthalene.

Phenols.

8. The distillate between 170° and 270° Centigrade, or 338° and 518° Fahrenheit (middle oils), shall not yield to caustic soda solution more crude tar acids (phenols) than is equivalent to 3 per cent by volume of the tar.

Free Carbon.

9. The tar shall contain not less than 12 per cent and not more than 21 per cent by weight of free carbon. The free carbon is to be determined by complete extraction of the bituminous matter from a weighed portion of the tar by benzol and bisulphide of carbon. The residue left on treatment with these extractives is to be taken as "free carbon."

Road Board Specification No. 5.

Specification for Tar No. 2.

General. 1. This tar is suitable for making tar macadam and also may be used for surface tarring in very hot weather when the road crust is exceptionally dry.

Heating. 2. For surface tarring, the tar should be heated to such a temperature that it will reach the road surface in a highly fluid condition. The necessary temperature to attain this end will vary with the mode of application of the tar. The desired temperature will be generally found in practice to be between 260° and 280° Fahrenheit or 124° and 138° Centigrade in the heater or "boiler." The tar should be heated in a heater or "boiler" specially designed to prevent frothing, which will otherwise inevitably occur if the tar contains even a small percentage of water.

For the preparation of tar macadam the tar will not generally need to be heated to so high a temperature as for surface tarring, but the necessary temperature should be determined largely by the sensible heat of the stone treated with the tar, and the mode of application or treatment.

Source of the tar. 3. The tar shall be derived wholly from the carbonization of bituminous coal except that it may contain not more than 25 per cent of its volume of the tar (or distillates or pitch therefrom) produced in the manufacture of carburetted water gas.

Specific gravity. 4. The specific gravity of the tar at 15° Centigrade (59° Fahrenheit) shall be as nearly as possible 1·21 and in no case shall it be lower than 1·19 or higher than 1·24.

Fractionation. 5. On distillation in a litre fractionating flask one-half to two-thirds filled, the tar should yield the proportions by weight of distillates stated below; the temperatures of distillation being read on a thermometer of which the bulb is opposite the side tube of the flask—

Below 170° Centigrade or 338° Fahrenheit, not more than 1 per cent of distillate (light oils and water if any).

Between 170° and 270° Centigrade or 338° and 518° Fahrenheit, not less than 12 per cent and not more than 18 per cent of distillate (middle oils).

Between 270° and 300° Centigrade or 518° and 572° Fahrenheit, not less than 6 per cent and not more than 10 per cent of distillate (heavy oils).

The total distillate between 170° and 300° Centigrade, or 330° and 572° Fahrenheit, shall be not less than 21 per cent and not more than 26 per cent, *i.e.*, where the middle oils approach the maximum allowed the heavy oils should approach the minimum allowed and *vice versâ.*

Naphthalene. 6. The distillate between 170° and 270° Centigrade, or 338° and 518° Fahrenheit (middle oils), shall remain clear and free from solid matter (crystals of naphthalene, etc.) when maintained at a temperature of 25° Centigrade for half an hour.

Phenols. 7. The distillate between 170° and 270° Centigrade, or 338° and 518° Fahrenheit (middle oils), shall not yield to caustic soda solution more crude tar acids (phenols) than is equivalent to 2 per cent by volume of the tar.

Free carbon. 8. The tar shall contain not less than 12 per cent and not more than 22 per cent by weight of free carbon. The free carbon is to be determined by complete extraction of the bituminous matter from a weighed portion of the tar by benzol and bisulphide of carbon. The residue left on treatment with these extractives is to be taken as "free carbon."

The testing of tar.

The compliance of a sample of tar with the Road Board Specifications for Tars No. 1 and No. 2 can be ascertained only in a chemical laboratory. But in cases where it is desired to ascertain in a quick and simple manner whether consignments of tar differ fundamentally from an approved sample, or whether they are of the No. 1 or of the No. 2 grade, the following simple tests may be of service:—

(1) Specific Gravity.—The specific gravity may be ascertained quickly and with a sufficient degree of accuracy by means of a hydrometer, to the readings of which a correction for deviation of temperature from the standard temperature of 15° Centigrade or 59° Fahrenheit is applied. A hydrometer for the range 1·16—1·24, which is wholly made in German silver, and will stand a reasonable amount of rough usage, is convenient, and this may be obtained in conjunction with a temperature corrector consisting of a substantial thermometer graduated to show directly the addition which should be made to the hydrometer reading when the temperature of the tar is above 15° Centigrade or 59° Fahrenheit. If a portion of the tar is poured into a suitable vessel, stirred, and the metal hydrometer and temperature corrector inserted in it, the specific gravity of the tar at 15° Centigrade or 59° Fahrenheit is obtained in two or three minutes even though the temperature of the tar at the time is considerably higher than 15° Centigrade or 59° Fahrenheit.

The specific gravity of a tar is not by itself a sufficient indication of the utility of the tar.

(2) Viscosity.—A standardized viscosimeter will show quickly whether a sample of tar is of the No. 1 or of the No. 2 grade, or whether a consignment differs fundamentally from an approved sample, but since the viscosity of tar varies greatly with its temperature it is necessary that readings of viscosity, in order to be comparable, should be made at the same temperature. The temperature of 25° Centigrade (77° Fahrenheit) is a convenient standard temperature for observations of the viscosity of tar, and it is necessary before using the viscosimeter that the tar should be exactly at this temperature and well stirred. In cases where serious disagreement is found between the viscosity of an approved sample and the viscosity of the tar as supplied, further examination of the latter should be made before it is used.

(3) Water and Naphthalene.—If about a quart of the tar is poured into a vessel about 12 inches high, and the vessel is covered by a piece of ordinary glass, and is left standing in a moderately warm room for 24 hours, flaky white crystals of naphthalene will be seen on the glass and the upper part of the walls of the vessel if there is a considerable amount of naphthalene in the tar. Globules of water also will be noticeable on the surface of the tar at the end of 24 hours if the tar contains a considerable quantity of water. Since most tars contain some naphthalene and often a trace of water also, it is advisable when making this test to put alongside the portion of tar which is being tested a portion of the approved sample in a similar glass covered vessel, and to compare the amount of naphthalene crystals deposited and of water separated from the two samples at the end of 24 hours.

Road Board Specification No. 6.

SPECIFICATION FOR PITCH.

1. This pitch is suitable for pitch-grouting. See "Road Board General Directions for Pitch-grouting."

2. The pitch is obtained of the required consistency most conveniently by running it off from tar stills in which the distillation of the tar has been stopped at the point at which the residual pitch will give a penetration of 70 (or such other penetration as may be specified to suit climatic or local conditions) when tested at 25° Centigrade (77° Fahrenheit) on a standard penetrometer. Harder pitch may be softened or cut back, in the still or in a mixer at the tar works, to the extent necessary for it to give this penetration, by the addition of tar oil of the grade specified below in Clauses 7 to 10.

Where pitch of the required consistency is not thus directly procurable, it may be prepared by softening commercial soft pitch, as specified below in Clauses 4 to 6, by the addition of tar oil as specified below in Clauses 7 to 10. In preparing the softened pitch in this manner the tar oil is added to the pitch in the manner described under "Instructions for Melting the Pitch" in the "Road Board General Directions for Surfacing with Pitch-Grouted Macadam," in such proportions that the resultant softened pitch will give a penetration of 70 (or such other penetration as may be specified to suit climatic or local conditions) when tested at 25° Centigrade (77° Fahrenheit) on a standard penetrometer, with a No. 2 needle weighted to 100 grammes for five seconds.

PREPARED PITCH FROM TAR DISTILLERIES.

3. Pitch which has been procured of the required consistency directly from a tar distillery needs only to be thoroughly melted in the pitch heaters or boilers, but as a precaution against burning, one to two per cent of tar oil may advantageously be put into the boilers with the pitch.

Pitch which has been procured of the required consistency directly from a tar distillery shall not yield more than 4 per cent of distillate below 270° Centigrade, or 518° Fahrenheit, on distillation described below in Clause 5, and shall contain not less than 16 per cent and not more than 28 per cent of "free carbon," as defined below in Clause 6.

COMMERCIAL SOFT PITCH.

4. The pitch shall be derived wholly from tar produced in the carbonization of bituminous coal, except that it may contain not more than 25 per cent of pitch derived from tar produced in the manufacture of carburetted water gas.

5. On distillation in a litre fractionating flask one-half to two-thirds filled the pitch shall yield the proportions by weight of distillates stated below; the temperatures of distillation being read on a thermometer of which the bulb is opposite the side tube of the flask:—

Below 270° Centigrade or 518° Fahrenheit, not more than 1 per cent of distillate.

Between 270° and 315° Centigrade or 518° and 599° Fahrenheit, not less than 2 per cent and not more than 5 per cent of distillate.

6. The pitch shall contain not less than 18 per cent and not more than 31 per cent by weight of free carbon. The free carbon is to be determined by complete extraction of the bituminous matter from a weighed portion of the pitch by benzol and bisulphide of carbon. The residue left on treatment with these extractives is to be taken as "free carbon."

Tar oil.

7. The tar oil to be used is preferably a filtered green or anthracene oil, and shall be derived wholly from tar produced in the carbonization of bituminous coal or from such tar mixed with not more than 25 per cent of its volume of tar produced in the manufacture of carburetted water gas.

8. The specific gravity of the tar oil at 20° Centigrade (=68° Fahrenheit) shall lie between 1·065 and 1·085.

9. The tar oil after standing for half an hour at 20° Centigrade (=68° Fahrenheit) shall remain clear and free from solid matter (naphthalene, anthracene, etc.).

10. The tar oil shall be commercially free from light oils and water. On distillation in a litre fractionating flask one-half to two-thirds filled, the tar oil shall yield the proportions by weight of distillates stated below: the temperature of distillation being read on a thermometer of which the bulb is opposite the side tube of the flask:—

Below 170° Centigrade or 338° Fahrenheit, not more than 1 per cent of distillate (light oils, and water, if any).

Below 270° Centigrade or 518° Fahrenheit, not more than 30 per cent of distillate (middle oils, and light oils and water, if any).

Below 330° Centigrade or 626° Fahrenheit, not less than 95 per cent of distillate (heavy oils, middle oils, and light oils and water, if any).

APPENDIX L.

Specification for oiling water-bound Macadam.

(Note on the method adopted at Delhi.)

The oil used was Fuel oil to which 5 per cent of coal tar was added. The crude oil is easily sprayed through a wheel valve under a head of three feet and the coal tar readily mixes with crude oil but quickly separates from it again owing to its greater specific gravity, the mixture must therefore be continually stirred up.

Before spreading the first coat the road surface must be thoroughly cleaned with brushes so as to remove all dust, etc., as the success of the oiling operation depends almost entirely on the surface material and quality of the original road surface. Assuming that the original work is good, the stiffest brooms obtainable should be used. A large party of men should go over the road two days before it is to be treated thoroughly cleaning the surface and gutters and thus showing up any loose patches or hollows that may need repair before oiling is commenced. On the day oiling is to be commenced, a party of 10 men or so, with brushes, should be ready to spread the oil over the road surface, while another party of 20 men proceed ahead and clean the surface once more. The crude oil and tar is then spread from the crown of the road towards the gutters. The first coat requires two days to dry, the second coat requires four, and the third seven to ten days. After three coats no watering should be required for a year.

In Delhi, the oil used was obtained from the Anglo-Persian Oil Company, Ltd., for whom Messrs. Shaw Wallace and Company, Calcutta, are the Agents. The cost of the oil landed in tank wagons of 3,528 gallons capacity, at Delhi, worked out at 8 annas per gallon, and the cost of coal tar was 14 annas per gallon.

Great care should be taken that foreign bodies do not become saturated with the crude oil and that where storage is necessary the tank should be hermetically sealed and no smoking allowed anywhere in its vicinity. It may be mentioned that a receptacle which is water-tight will not necessarily be oil-tight, and extra care in riveting is necessary when making tanks for storage therefor, and any iron tanks employed should be riveted metal to metal, and caulked with proper caulking tools.

The coal tar used is the ordinary kind obtainable in the market, and it is advocated that an admixture of 5 per cent of tar should be made to all three coats. The first coat requires by far the biggest quantity of oil as it soaks into all crevices. The amount of coal tar actually used on an average was 10 tons for the first coat, 5 tons for the second coat, and 4 tons for the third coat, in the case of a 12-foot road one mile long.

At Delhi, the oil, which was railed from Budge Budge in oil tanks, was emptied out by gravity into masonry storage tanks of about 1,200 gallons capacity. From the latter the oil was pumped into two iron tanks each of 600 gallons capacity, which were fitted on to a steam tractor. At the back of each tank was a $2\frac{1}{2}$ in. wheel valve from which the oil was drawn off when required. There were openings in the top of the tanks to allow of stirring up of the mixture of oil and coal tar.

If possible, the traffic should be diverted along other roads during the process of oiling, but if, as at Delhi, this cannot be done, it is necessary to

have warning signals, especially during the second and third coating as the oily surface is very slippery and dangerous to traffic until absorbed. After the oiling is completed, the surface of the road should be maintained in the usual way with the exception that no watering is required.

PUNJAB SPECIFICATION FOR TAR SPRAYING ON WATER-BOUND MACADAM.

Tarring will only be done in warm weather when a road is perfectly dry, and after all ruts and pot holes have been filled and other necessary repairs to the surface of the macadam completed.

The surface must be swept clean from all dust and any caked mud (when resurfacing is necessary, as a preliminary to tarring, only stone chippings, without any finer material, should be used for binding), and when only half the width of a road can be treated at a time, the dust should also be removed from the adjoining portion to prevent its being stirred up by passing traffic.

Coal tar (tar derived wholly from the carbonization of coal), commercially free from water, and containing not more than 10 per cent of its volume of the tar produced in the manufacture of carburetted water gas, will then be heated to its boiling point(220° to 240° F.), at convenient positions on the work, and applied as hot as possible so that it will flow freely and can be brushed in an even film over the surface. The heating must be done in a boiler specially designed to prevent the frothing which will otherwise inevitably occur if the tar contains even a small percentage of water. The quantity of tar required will vary according to the physical conditions of the road, but, on a first application, a gallon should suffice for five to seven superficial yards.

Grit, consisting of stone chippings, crushed gravel, or coarse sand, free from dust, and not larger than will pass through a ½ in. mesh, should be spread on the surface next day, but only in sufficient quantity to prevent the tar from adhering to the wheels.

Where traffic is heavy, a second coat (one gallon sufficing for eight to ten yards super) is advisable, at least over the centre portion of the road, two or three months after the first application, and surface tarring should be renewed annually on all important roads, and as required on roads with light traffic.

APPENDIX LI.

Specification for Bituminous or Asphalt Macadam.

(Excluding Tars and Pitches.)

1. *Definition of Bitumen for Road Purposes.*—Bitumen is a generic term for a group of hydrocarbon products soluble in carbon disulphide, which either occur in nature or are obtained by the evaporation of asphaltic oils. The term should not include residue from paraffin oil or coal tar products.

2. *Definition of Asphalt.*—Asphalt is a material consisting of a mixture of bitumen and finely graded mineral matter. The mineral matter may range from an impalpable powder up to material of such size as will pass through a sieve having square holes of ¼ in. side.

SURFACING EXISTING ROAD WITH BITUMINOUS MACADAM.

(2 coat work.)

3. The existing road surface should be lightly scarified and re-formed, and should be steam-rolled and consolidated to the contour required for the new surface.

4. *Subcrust, Binder Course, or Bottom Layer.*—The subcrust should consist of hard, clean, broken stone, 1 in gauge, the voids of which are filled with ⅜ in. size chippings, while the voids in the mixed stone should be filled with well-graded sand, sufficient bitumen and filler being added to thoroughly coat the mineral aggregate without showing any excess on compression with a hot tamper.

5. The materials should be mixed in proportions by weight, depending upon their character. The proportions will ordinarily vary between the following limits :—

1 in. gauge stone . .	56 to 30 per cent.
⅜ in. size chippings .	16 to 26 ,,
Graded sand . .	25 to 33 ,,
Filler (Portland cement)	3 to 5 ,,
Bitumen . . .	6 to 8 ,,

6. The aggregate and bitumen should be heated separately to such temperatures that the finished mixture shall, depending on the bitumen used, have a temperature of from 300° to 350° F.

7. The subcrust should be spread while hot upon the foundation to such depth that, after being immediately consolidated by rolling, its average thickness shall be 2½ in., and its upper face graded to the contour required for the wearing surface.

8. As an alternative the subcrust may be composed of material coated with tar or grouted with pitch mixture, laid in accordance with the Road Board Specifications Nos. 2 and 3 respectively.

9 *Wearing Surface.*—The mixture for the wearing surface should be composed of :—

(*a*) Bitumen.
(*b*) Sand of satisfactory grading.
(*c*) Filler, consisting of finely powdered mineral matter.

10. *Sand.*—The sand should consist of hard grains, not necessarily sharp, of the following grading:—

	Heavy Traffic Per cent.	Light Traffic Per cent.
Passing 80 mesh and retained on 200 mesh .	34	20
Passing 40 mesh and retained on 80 mesh .	43	45
Passing 10 mesh and retained on 40 mesh .	23	35

11. *Filler.*—The filler shall consist of suitable material ground to an impalpable powder. Portland cement is generally used.

12. *Combining Materials.*—The materials should be mixed in proportions by weight depending upon the character of the materials. The proportions will ordinarily vary between the following limits:—

Bitumen	11 to 12 per cent.
Filler	13 to 15 ,,
Graded sand	73 to 76 ,,

The percentage of matter soluble in carbon disulphide in any mixture shall not be less than 9·5 per cent or more than 13 per cent.

13. The sand and bitumen should be heated separately to such temperatures that the finished mixture shall, depending upon the bitumen in use, have a temperature of from 300° to 350° F. The filler should be mixed while cold with the hot sand. The bitumen will then be mixed with the sand and filler at the required temperature and in the proper proportion in a suitable apparatus, so as to effect a thoroughly homogeneous mixture.

14. *Laying.*—The mixture should be carted on to the road in trucks, properly protected from radiation by tarpaulins, at a temperature of not less than 250° F., and spread upon the subcrust to such depth as will ensure an average thickness of 1½ in. after ultimate compression.

15. The compression will be attained by first smoothing the surface with a hand roller working from kerb to centre, after which stone dust should be swept over it, when rolling will be continued with a light steam or petrol roller until the surface is properly consolidated.

APPENDIX LII.

Specifications for cement concrete and reinforced concrete road foundations and surfacing.

REINFORCED CONCRETE ROAD FOUNDATIONS.

1. The advantages of reinforcing cement concrete road foundations, the use of which has developed to meet the exigencies caused by the increased weight and increased speed of motor traffic, are that great strength is obtained with a small thickness of concrete, and that the construction is resilient. Concrete 6 in. thick was first adopted with a light reinforcement, but the tendency has been with the advent of heavier traffic to increase both the thickness of concrete and amount of reinforcement. The cost of the steel approximates to about 2 in. of concrete, whereas a slab 8 in. thick suitably reinforced is stronger than 15 in. of unreinforced concrete.

2. At the same time the reinforcement gives tensile strength to the concrete, and in the case of double-layer reinforcement shear stress also, and provides some degree of resilience. It is probable that further experience may show a lesser thickness of reinforced concrete to be sufficiently strong and more resilient.

3. Wire fabric or other reinforcement is generally laid with its main wires, or members, parallel to the length of the road, although the more correct method is to provide for a reinforcement with the same sectional area of steel transversely and longitudinally.

4. The reinforcement is laid about 1½ in. to 2 in. above the bottom of the concrete. This is convenient in practice and correct in theory, the condition being similar to that of a cantilevered floor-slab turned upside down, the wheel taking the place of the support and the earth pressure taking the place of a distributed load of intensity greatest at the support and gradually reducing outwards. The reinforced concrete foundation is sufficiently strong to spread the very heaviest wheel-load over so large an area of underbed that the pressure becomes less than one ton per square foot, so that even in bad ground there is no subsidence of the foundation, and no hollows form in the surface except from actual surface wear.

CEMENT CONCRETE AND REINFORCED CONCRETE ROAD SURFACING.

1. In some cases plain concrete is used, elsewhere it is reinforced. Transverse joints ½ in. thick filled with bituminous material were first adopted 30 ft. apart to allow for expansion and contraction, but these joints appear to have been unsatisfactory, and are more or less a source of weakness. The later tendency is for the concrete surfacing to be carried out on the "alternate bay" principle, and to reinforce the concrete, by which means cracks due to setting contraction are eliminated.

2. Usually the concrete is 7 in. thick. The reinforcement is generally wire fabric or expanded metal, or double-layer reinforcement. This not only does away with the joints but gives a very much stronger road than the lesser thickness first adopted, and is from every point of view a better form of construction.

3. The proportions of concrete that have given the best results are: Two-course work, lower course seven parts of coarse and fine aggregate to

one of cement, and three parts of fine aggregate to one of cement for the surface course. Successful results have also been obtained by the use of large tough aggregate (preferably granite) in the top course, which provides an excellent key for the waterproofing surface layers, the proportions of the concrete in this layer being 3 parts of aggregate to $1\frac{1}{2}$ sand and 1 of cement.

4. A great deal of attention has been paid to the question of waterproofing and hardening of concrete roads and several proprietary preparations having these objects in view are successfully employed for these purposes, not only in Great Britain, but, to a far greater extent, in America.

5. The average maintenance cost of concrete roads in the United States is one-fifth of a penny per square yard per annum. The roads may be surface-dressed with tar and granite chips, which is a precaution against such small cracks as may occur owing to lack of care in joining up one day's work to the next, or in insufficient watering of the surface during construction. Cracks from these causes may be such as to just admit the blade of a knife, but where the roads are tar-sprayed the tar bridges over the cracks and makes them of no account.

6. Concrete roads are laid with a camber of one in fifty, which is sufficient to throw off water. This small slope almost eliminates side-slip.

APPENDIX LIII.

Road Construction abstract estimate form.

(To be modified as necessary to suit local requirements.)

Abstract Estimate of the cost of Constructing No.——Section (——to——)——Road. (Length——Miles.)

Serial No.	Item.	Sub-Item.	Detail.	Unit of calculation.	No. or quantity.	Rate.	Per	Cost.	Amount.	REMARKS. (Explanation of rates and costs and reference to appended details.)
						Rs.		Rs.	Rs.	
I	Road formation	(a) Excavation and filling earth.	Approximate quantity	C. ft.	..	..	0/00	..		
		(b) Excavation and filling gravel.	Do.	"	..	..	0/00	..		
		(c) Excavation and filling small boulders and gravel.	Do.	"	..	..	0/00	..		Based on detailed survey.
		(d) Excavation and filling large boulders and gravel.	Do.	"	..	..	0/00	..		Explanation of quantities and rates Annexure A.
		(e) Excavation and filling soft rock.	Do.	"	..	..	0/00	..		
		(f) Excavation and filling hard rock.	Do.	"	..	..	0/00	..		
		(g) Excavation and filling very hard rock.	Do.	"	..	..	0/00	..		
		Etc.							..	
II	Retaining walls	(a) Dry stone average height X′.	Approximate length	R. ft.	..	..	0/0	..		Based on rough detailed type estimate Annexure B.
		(b) Dry stone average height Y′.	Do.	"	..	..	0/0	..		Based on rough detailed type estimate Annexure C.
		(c) Stone in lime Z′	Do.	"	..	..	0/0	..		
		Etc.							..	

APPENDIX LIII—*contd.*

Serial No.	Item.	Sub-item.	Detail.	Unit of calculation.	No. or quantity.	Rate.	Per	Cost.	Amount.	REMARKS. (Explanation of rates and costs and reference to appended details.)
						Rs.		Rs.	Rs.	
III	Training works	(a) Place	Average section .. × ..	R. ft.	..	..	0/0	..		Based on rough detailed type estimate Annexure D.
		(b) Place	Do.	„	..	..	0/0	..		
		Etc.							..	
IV	Drains	(a) Side drains	Approximate length	„	..	..	0/0	..		Based on rough detailed type estimate Annexure E.
		(b) Catchwater drains	Do.	„	..	..	0/0	..		Based on rough detailed type estimate Annexure F.
		Etc.							..	
V	Scuppers	(a) X′ span	Stone in lime	Scupper	..	..	each	..		Based on rough detailed type estimate Annexure G.
		(b) Y′ span	Cement concrete	„	..	..	„	..		Based on rough detailed type estimate Annexure H.
		Etc.							..	
VI	Culverts	(a) 3′ tube	C. I. tube	Culvert	..	..	each	..		Based on rough detailed type estimate Annexure I.
		(b) 2′×2½′ tube	R. C. tube	„	..	..	„	..		
		(c) X′ span	Masonry arch	„	..	..	„	..		Based on rough detailed type estimate Annexure——.
		(d) Y′ span	R. C. slab	„	..	..	„	..		Based on rough detailed type estimate Annexure——.
		Etc.							..	

APPENDIX LIII—*contd.*

Serial No.	Item.	Sub-Item.	Detail.	Unit of calculation.	No. or quantity.	Rate.	Per	Cost.	Amount.	REMARKS. (Explanation of rates and costs and reference to appended details.)
						Rs.		Rs.	Rs.	
VII	Dressing and rolling formation.	(*a*) Dressing and ramming.	(Length) × (Average width)	Sq. ft.	..	..	0/0	..		Based on rough detailed type estimate Annexure——.
		(*b*) Dressing and rolling	Do.	,,	..	..	0/0	..		Based on rough detailed type estimate Annexure——.
		Etc.							..	
VIII	Road parapets	(*a*) Dry stone	Approximate length	R. ft.	..	..	0/0	..		Based on rough detailed type estimate Annexure——.
		(*b*) Stone in lime	Do.	,,	..	..	0/0	..		Based on rough detailed type estimate Annexure——.
		Etc.							..	
IX	Metal stacking places.	(*a*) Excavation and filling large boulders in gravel.	Approximate quantity	C. ft.	..	..	0/00	..		
		(*b*) Excavation and filling large boulders in soft rock.	Do.	,,	..	..	0/00	..		Approximate detailed measurements Annexure——.
		(*c*) Excavation and filling large boulders in hard rock.	Do.	,,	..	..	0/00	..		
		Etc.							..	
X	Soling	(*a*) Collection	(i) Length × width × thickness.	C. ft.	..	..	0/00			Explanation of rates Annexure——.
			(ii) Length × width × thickness (corners).	,,	..	..	0/0	..		
			(iii) Length × width × thickness (on embankment).	,,	..	..	0/0	..		
		(*b*) Laying and dry consolidation.	(i) For collection	,,	..	..	0/0	..		Explanation of rates Annexure——.
			(ii) Do.	,,	..	..	0/0	..		
			(iii) Do.	,,	..	..	0/0			
			Etc.						..	

APPENDIX LIII—*contd.*

Serial No.	Item.	Sub-item.	Detail.	Unit of calculation.	No. or quantity.	Rate.	Per	Cost.	Amount.	REMARKS. (Explanation of rates and costs and reference to appended details.)
XI	Metalling .	(a) Collection . .	(i) Length × width × thickness.	C. ft.	..	Rs. ..	0/0	Rs. ..	Rs.	Explanation of rates Annexure——.
			(ii) Length × width × thickness (corners).	„	..	..	0/0	..		
			Etc.							
		(b) Laying and consolidation.	(i) Ledgth × width × thickness.	„	..	..	0/0	..		Ditto.
			(ii) Length × width × thickness (coners).	„	..	..	0/0	..		
			Etc.							
		(c) Dressing and rolling Berms.	Length × average width × thickness.	„	..	..	0/0	..		Ditto.
		Etc.							..	
XII	Causeways . .	(a) Place . . .	Length and type . .	R. ft.	..	..	R. ft.	..		Based on rough detailed type estimate Annexure——.
		(b) Place . . .	Do. . .	„	..	..	„	..		
		Etc.							..	
XIII	Bridges . .	(a) Place . . .	Length and type . .	R. ft.	..	..	R. ft.	..		Based on rough detailed type estimate Annexure——.
		(b) Place . . .	Do. . .	„	..	..	„	..		
		Etc.							..	
XIV	Road Signs .	(a) Mile stones . .	..	Stone	..	..	each	..		Based on rough detailed type estimate Annexure——.
		(b) Furlong stones .	..	„	..	..	„	..		Based on rough detailed type estimate Annexure——.
		(c) Warning signs .	..	Sign	..	..	„	..		Based on rough detailed type estimate Annexure——.
		(d) Direction posts .	..	„	..	..	„	..		Based on rough detailed type estimate Annexure——.
		Etc.							..	

APPENDIX LIII—*contd.*

Serial No.	Item.	Sub-item.	Detail.	Unit of calculation.	No. or quantity.	Rate.	Per	Cost.	Amount.	REMARKS. (Explanation of rates and costs and reference to appended details.)
						Rs.		Rs.	Rs.	
XV	Siding and parking places.	(a) Sidings	(i) Place and size	..	..	..	L. S.	..		
			(ii) Do.	..	..	..	L. S.	..		
			Etc.							
		(b) Parking places	(i) Place and size	..	..	..	L. S.	..		
			(ii) Do.	..	..	..	L. S.	..		
			Etc.						..	
XVI	Land compensation.	(a) Acquisition of land	Approximate detail attached.	..	..	..	..	..		Assessed in conjunction with local civil officer. Approximate detail Annexure——.
		(b) Compensation for crops, graves, trees, water mills, irrigation channels.	Do.	..	..	..	..	..		
		Etc.							..	
XVII	Accommodation	(a) Temporary offices and accommodation for staff, labour and stores.	(i) Description and place	..	..	..	L. S.	..		Approximate plinth area, or cube area estimate Annexure——.
		(b) Rest house	(ii) Do.	..	..	..	L. S.	..		
			Etc.							
			(i) Description and place	..	..	..	L. S.	..		Approximate plinth area, or cube area estimate Annexure——.
		Etc.	Etc.						..	
			Deduct—For credits by disposal of materials on dismantlement of temporary buildings.				L. S.	..	..	

APPENDIX LIII—*concld.*

Serial No.	Item.	Sub-item.	Detail.	Unit of calculation.	No. or quantity.	Rate.	Per	Cost.	Amount.	REMARKS. (Explanation of rates and costs and reference to appended details.)
						Rs.		Rs.	Rs.	
XVIII	Water supply .	(*a*) Place . . .	Description . . .	..	..	..	..	..		Approximate estimate of cost Annexure——.
		(*b*) Place . . .	Do. . . .	..	..	..	..	..		Approximate estimate of cost Annexure ——.
		Etc.								
			Deduct—Credits by disposal of materials on dismantlement of temporary installations.				L. S.	..	..	
							TOTAL	..	..	
XIX	General contingencies.	Including works charge establishment, flood damage repairs during construction and special tools and plant.	5% on total of items I to XVIII (increased up to 10% where essential).	..	..	..	..	..	..	
			Deduct—For credits by disposal of tools and plant on completion of work.				L. S.	..	..	
							TOTAL	..	..	
XX	Political charges .	(*a*) Protection . .	(Particulars) . . .	..	..	..	..	..		Assessed by or in conjunction with local civil officer.
		(*b*) Royalties . .	Do. . . .	..	..	..	..	..		
		(*c*) Land compensation .	Do. . . .	..	..	..	..	..		
		(*d*) Miscellaneous	Do. . . .	..	..	..	..	..		
						GRAND TOTAL		..	..	

APPENDIX LIV.

Bridge construction abstract estimate form.

(To be modified as necessary to suit varying types of Bridge and local requirements.)

ABSTRACT ESTIMATE OF CONSTRUCTING THE..........................BRIDGE AT................

Serial No.	Item.	Sub-item.	Detail.	Cost.	Amount.	REMARKS. (Explanation of rates and costs and reference to appended details.)
				Rs.	Rs.	
I	Training works .	(*a*) Right Bank Training Bund.	Description	..		Approximate detail Annexure A.
		(*b*) Left Bank . .	Do. . . .	..		
		Etc.			..	
II	Piers and Abutments.	(*a*) Piers	Excavation and Masonry .	..		
		(*b*) Right Bank Abutment and wing walls.	Do. .	..		
		(*c*) Left Bank Abutment and wing walls.	Do. .	..		
		(*d*) Unwatering foundation.	Do. .	..		
		(*e*) Shoring and centering.	Do. .	..		Approximate detail Annexure B.
		Etc.			..	

APPENDIX LIV—*contd.*

Serial No.	Item.	Sub-item.	Detail.	Cost.	Amount.	REMARKS. (Explanation of rates and costs and reference to appended details.)
				Rs.	Rs.	
III	Bridgehead Defences	(*a*) Right Bank Abutment	Description . . .	..		
		(*b*) Left Bank Abutment	Do. . . .	..		
		Etc.				
					..	
IV	Supply of superstructure Steelwork.	(*a*) Girder Spans . .	Description and Number .	..		
		(*b*) Steel Troughing .	Description . . .	..		
		(*c*) Carriage . .	From......to...... . .	..		Approximate detail Annexure C.
		Etc.				
					..	
V	Erection of superstructure Steelwork.	(*a*) Carriage . .	From......to...... . .	..		
		(*b*) Erection . .	..	..		Approximate detail Annexure D.
		Etc.				
					..	
VI	Bridge Roadway .	....	Description . . .	..		Approximate detail Annexure——.
					..	

APPENDIX LIV—*contd.*

Serial No.	Item.	Sub-items	Detail.	Cost.	Amount.	REMARKS. (Explanation of rates and costs and reference to appended details.)
				Rs.	Rs.	
VII	Approaches . .	(*a*) Right Bank Approach Road.	Description . . .	..		
		(*b*) Left Bank Approach Road.	Do. . . .	..		
		(*c*) Subsidiary Bridge on Left Bank Approach.	Description and length .	..		Approximate detail Annexure——.
		Etc.			..	
VIII	Land compensation	(*a*) Acquisition of Land.	Approximate detail attached.	..		Assessed in conjunction with local civil officer.
		(*b*) Compensation for crops.	Do. .	..		Approximate detail Annexure——.
		Etc.			..	
IX	Accommodation .	(*a*) Temporary stores godown.	Description . . .	..		
		(*b*) Temporary quarters for staff.	Do. . . .	..		Approximate detail Annexure——.
		Etc.			..	
		Deduct for credits by disposal of materials on dismantlement of buildings.		..		
					..	

APPENDIX LIV—*concld.*

Serial No.	Item.	Sub-item.	Detail.	Cost.	Amount.	REMARKS. (Explanation of rates and costs and reference to appended details.)
				Rs.	Rs.	
X	Establishment .	....	Works Stores, and mechanical supervising and office establishments.	..		Approximate detail Annexure——.
					..	
XI	Special Tools and Plant.	(*a*) Pumping Plant .	Description . . .	..		
		(*b*) Steelwork erection Plant.	Do. . . .	..		
		(*c*) Miscellaneous Tools and Plant.	Do. . . .	..		
		Etc.				
					..	Approximate detail Annexure——.
		Deduct for credits by disposal of Tools and Plant on completion of work.		..	..	
			TOTAL	..	..	
XII	General contingencies.	Including flood damage repairs during construction.	5% on total of items I to XI (increased to 10% where essential).	..	..	
			TOTAL	..	..	
XIII	Political charges .	(*a*) Protection . .	(Particulars) . . .	..		Assessed by or in conjunction with local civil officer.
		(*b*) Royalties . .	Do. . . .	..		
		(*c*) Land compensation .	Do. . . .	..		
		(*d*) Miscellaneous . .	Do. . . .	..		
			GRAND TOTAL .	..	..	

SPECIAL CHARGES IV.

Item.	Description.	Abstract of cost or L. S. Estimate for whole road.	Remarks. Enter explanation for any notable variations from amounts last sanctioned in Form I (excluding Form II) and any other useful explanation against each item. A statement should be appended giving the names, numbers, and capital costs of the buildings concerned, also numbers and current wages of each class of personnel.
		Rs.	
1	Maintenance and repairs of Boat Bridges.		
2	Special repairs to Buildings .		
3	Clearing Slips		
4	Masonry repairs to Bridges, Culverts, Walling, *Pukka* Drains and Causeways.		
5	Painting Ironwork of Bridges		
6	Whitewashing and painting Mile Posts, Furlong Stones, Road Signs, numbers on Culverts, etc.		
7	Training works of Bridges .		
8	Miscellaneous		
	TOTAL .		Average per mile =

ARBORICULTURE V.

1	Malis		
2	Other Staff		
3	Watering arrangements .		
4	Protecting Trees . . .		
5	Miscellaneous . . .		
	TOTAL .		Average per mile =

ABSTRACT OF ESTIMATE.

No. of Sub-head.	Description of Sub-head.	Average rate per mile.	TOTAL.
		Rs.	Rs.
I	Maintenance (Ordinary) . .		
II	Remetalling		
III	Maintenance (Special) . .		
IV	Special charges		
V	Arboriculture (see Note 4) .		
	TOTAL .		

NOTES.

1. Use a separate form for each main road.
2. Cantonment roads may be grouped, but state classification in column 1 under I and II.
3. Sub-division into Sections depends on headings of Columns 2 and 3 and other factors under I and II.
4. When Arboriculture in a civil charge, omit Sub-head V from military road estimates.
5. The cost of Road Roller repairs and salaries of Sub-Overseers are not legitimate charges against road maintenance.

Prepared by ——————Checked by ——————Signed by ——————

Designation ——————Designation ——————*Assistant Commanding Royal Engineer.*

Cost debitable to { Repairs, Military. / Repairs, Civil Communications. / Miscellaneous Public Improvements.

Date ——————

Forwarding No. ——————

APPENDIX LV.

FORM I.

Estimate of Average Cost of Road Maintenance.

Sub-District ——————————Year 192 ——————192

Name of Road ——————————Total Length ——————miles.

Item.	Section. Give mileage, or in case of Grouped Cantonment Roads describe otherwise.	Estimated life of metal.	Width of metal.	Length.	Maintenance (Ordinary) I. Average cost per mile. Road gangs.	Patch metal.	Total per Mile. Columns 5+6	Total per Section. Columns 4×7	Remetalling II. Average cost per Mile.	Amount cost per Mile. Columns 9+2	Section. Columns 10×4
	1	2	3	4	5	6	7	8	9	10	11
No.		Years.	Feet.	Miles.	Rs.	Rs.	Rs.	Rs.	Rs.	Rs.	Rs.
	Total .										

MAINTENANCE (SPECIAL) II.

Item.	Description.	Abstract of Cost or L. S. estimate for whole road.	Remarks. Enter explanations for any notable variations from amounts last sanctioned in Form I (excluding Form II) and any other useful explanation against each item. A statement should be appended giving the names, numbers, and capital costs of the buildings concerned, also numbers and current wages of each class of personnel.
		Rs.	
1	A. I. R. to Buildings . .		
2	Chaukidars		
3	Work Munshies or Mates .		
4	Khalasis		
5	Clearing Causeways . .		
6	Shingling		
7	Miscellaneous . . .		
	Totals .		Average per mile =

APPENDIX LVI.

FORM 2.

ABSTRACT ESTIMATE OF REMETALLING (SUB-HEAD II OF FORM 1).

Sub-District...year 192 -192

Name of Road..Total length........Miles.

Item.	Mileage.	Year and month when last remetalled, thus 1917-11	Estimated life of metal from Form 1, column 2.	Width of metal.	Nature of metal (*see* Note 1).	Nature of repair proposed (*see* Note 2)	RATES FOR (SEE NOTE 3).					Improvements as per detail attached	Total cost per mile.	Total of this estimate = Rs. Total from road maintenance estimate form, Sub-head II, *i.e.*, average cost of re-metalling . . . = Rs. Difference Saving in black / Excess in red.
							Metal Collection	Scarifying inches deep	Consolidation	Watering	Dressing Berms			
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
No.	From to	Date.	Years.	Feet.			%c ft.	%s.ft	%c ft	%c.ft	%s ft	Rs.	Rs.	NOTES.

NOTES.

1. Column 6—
   Boulders, limestone . = B.L.
   „ sandstones = B.S.
   „ mixed . . = B.M.
   Quarried limestone . . = Q.L.
   „ sandstone . = Q.S.
2. Column 7—
   New 4½" layer . . = R. (1)
   Scarifying *plus* 2" layer = S.+2"(2)
   Scarifying without fresh metal . . . . = S. (3)
   Continuous out repairs . = C.R.R. (4)
   Collection only for ( ) above = Coll. ( )
3. Columns 8 to 12. Enter estimated rates, not schedule rates.
4. Column 13. May include allowance for banking where not previously done, widening metal at curves or bridges to bring up to specification but nothing which is essentially a new work.
5. Metal collection includes all costs up to and including stacking at site.
6. Scarifying. This rate includes screening and stacking ready for further use. If scarified metal is to be rolled into berms include cost of this under column 12 and explain in covering letter.
7. Consolidation. Includes all operations from the stacks to completion, including cost of diverting traffic, but excluding watering.
8. Watering. Covers costs for all water used during consolidation for all purposes.
9. Dressing berms. See specifications for berm maintenance.

Prepared by..............Checked by..............Forwarded for sanction and allotment of funds on Form No. 1.
Designation..............Designation..............Sent under my No.........dated

Cost debitable to { Repairs, Military. / Repairs, Civil Communications. / Misc. Public Improvements.

Signed by..............
Asstt. Commanding Royal Engineer.
Date............Forwarding No.

APPENDIX LVII.

WIRE MATTRESSES FOR RIVER TRAINING WORKS.

Manufacture of wire nets in general.

1. *Tools.*—One pair wire cutter and one 2 lbs. Hand Hammer. For closing the net when filled, two Tommy bars, 15″ to 18″ long, ½″ to 1″ diameter, will also be needed.

2. *Plant.*—One peg block (A Fig. 4), one stretching block (B Fig. 4) and two squatting boards (Fig. 5).

3. *Peg block.*—15″ to 18″ wide, 4″ to 6″ thick and one feet longer than the finished width of the net. It is studded with 5″ wire nails spaced in 3 rows 4″ apart nail to nail and 4″ between rows, staggered as shown in Fig. 4. The heads of the nails are cut off, leaving pegs projecting 2″ above board.

4. *Stretching block.*—Any old sleeper will do. Some weight is necessary to keep the knots taut against the pegs and, incidentally, for the workmen to squat upon.

5. *Squatting board.*—4″ × 2″ and same length as peg block with 1″ auger holes to correspond with pegs. These boards are necessary to protect the men's feet when working on the outer row of pegs.

6. *Labour.*—One pair of coolies is needed for every 8 or 10 feet of width of net. Both should know each other's job as the actual twisting of the wires is hard on the hands, and if equally trained the two men change places at intervals.

7. *Manufacture.*—(A) The wire is uncoiled, stretched along the ground and cut to the required lengths. The precise lengths can best be determined by making an experimental net and noting the lengths of wire used (*vide* Fig. 10 for Standard Nets). After cutting all wires of required length each is doubled at its centre and both ends coiled back to the centres for convenience of handling. When the full number of wires have been cut and coiled, *viz.*, one double wire of two coils for every 4″ width of netting *plus* one for the end (*e.g.*, a 3′ width of net would require $(36 \div 4) + 1 = 10$ double wires of 2 coils each), a loop is formed at the centre as each wire is taken into use against the first row of pegs as shown at Figs. I and II. A squatting board is then placed over the loops and the coils are opened and laid loose over the peg board as indicated in Fig. 4-A.

(B) One man of the pair squats behind the peg board and the other handles the coils. One of the two wires of loop No. 1, Fig. I, and one of loop No 2 are twisted together against the first peg in the second row of pegs, Fig. 4-A, by the man at the board while the man handling the coils turns the coils to correspond with the twists. The twists are made up against the pegs as shown in Fig. 3 and are then—if necessary—flattened by a blow or two from the hammer against the peg block. The other wire of No. 2 loop, Fig. I, is twisted with the first wire of No. 3 loop against the second peg and so on until the whole width has been completed. All twists must be made in the same direction.

(C) The second squatting board is now placed over the second row of pegs and the wires twisted against the third row of pegs. The third row having been completed the squatting boards are removed, the net taken off the pegs, pulled back and the last made twists placed over the first row of pegs. The net projecting behind the peg block is placed under the stretching